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Flood prediction and warning program for Squaw Creek at Ames, Iowa

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by

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INTRODUCTION

History of Flooding Along Squaw Creek

The Squaw Creek drainage basin lies in northwestern Story, northeastern Boone, and southwestern Hamilton counties in Central Iowa (Figure 1). Land use in this 227 square mile region is primarily row crop agriculture. Corn and soybeans are the predominant crops.

The only major urban area in the basin is at its southern tip. Here, Squaw Creek flows for approximately four miles through the campus of Iowa State University and the city of Ames before joining the Skunk River near the southeastern boundary of the city.

Gaging station data and other historical records for Squaw Creek show that major floods have occurred at least eight times since 1918, and that the designated flood stage of seven feet has been exceeded on at least 52 occasions since that time. Table 1 summarizes the occurrence of these events.

On the basis of the damage it caused, the flood of 1975 was, by far, the most spectacular one in the basin to date. According to Lara and Heinitz (U.S. Dept. of the Interior 1976), the 1975 flood on Squaw Creek was the most damaging of any flood in the Skunk River basin, a 4,355 square mile drainage area extending from northcentral Iowa to the

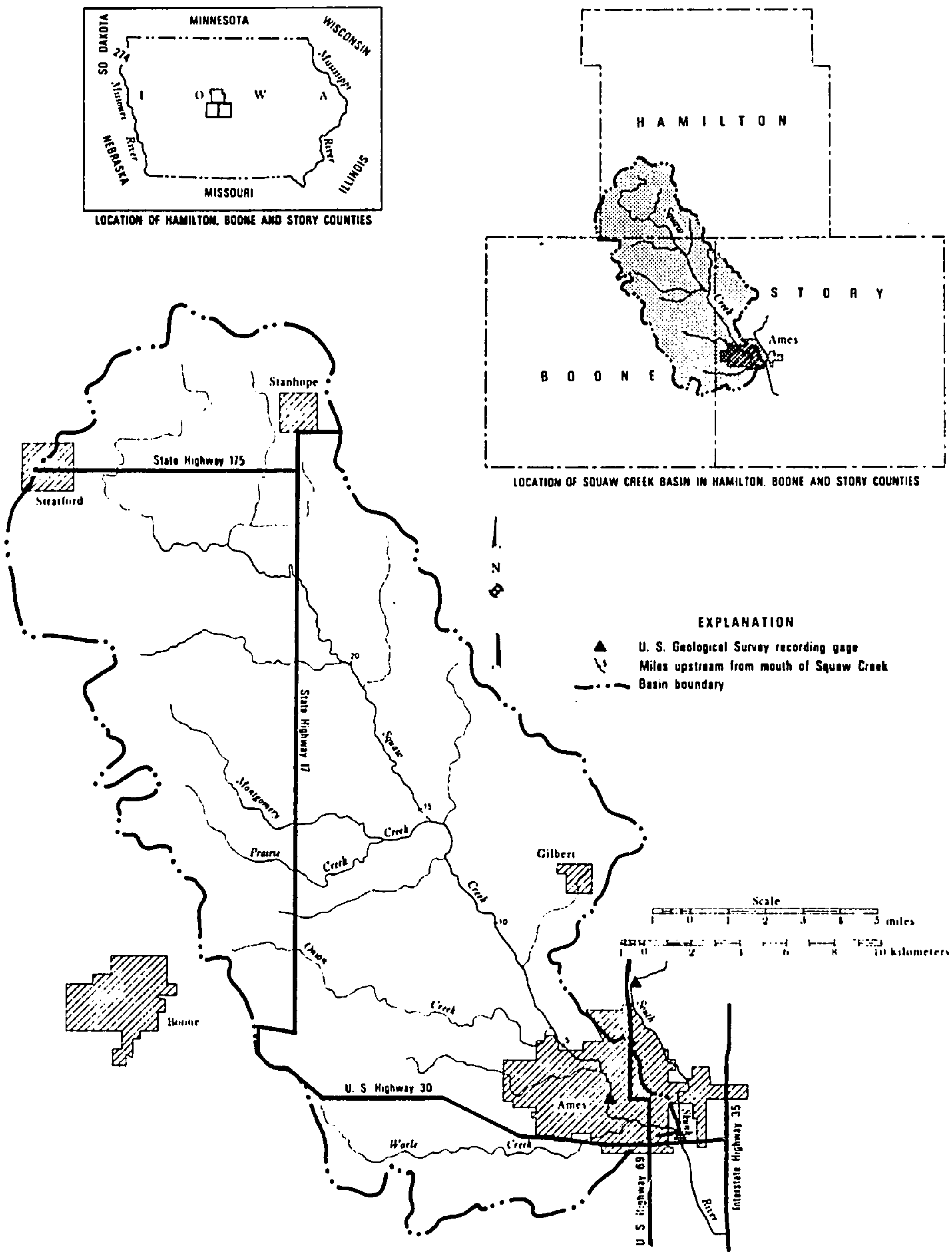

Figure 1. Map of Squaw Creek basin (U.S. Dept. of the Interior 1976)

Table 1. Major floods in the Squaw Creek basin (U.S. Dept. of Interior 1978, 1979, 1980, 1981, 1982, 1983, 1984, 1985, 1986)

Date	Gage Height[a] (feet)	Discharge (cubic feet/second)
June 4, 1918	14.5	6,900
Sept. 30, 1919	7.96	1,900
Oct. 4, 1919	8.6	2,260
Sept. 17, 1921	7.4	1,900
July 17, 1922	10.7	4,130
July 28, 1924	8.8	3,170
Sept. 19, 1926	10.2	3,610
May 20, 1944	—[b]	—[c]
June 13, 1947	—[b]	—[c]
June 1, 1954	—[b]	—[c]
August 28, 1954	—[b]	—[c]
July 5, 1958	—[b]	—[c]
March 30, 1960	—[b]	—[c]
March 1, 1965	10.70	4,200
June 4, 1965	8.85	2,680
June 12, 1966	10.15	3,160
June 25, 1968	8.27	2,500
March 20, 1969	9.59	2,970
March 24, 1969	7.16	2,120
June 7, 1969	8.34	2,240
June 30, 1969	9.45	2,580
July 9, 1969	7.84	2,090
May 13, 1970	10.74	3,540
Feb. 19, 1971	10.09	3,650

[a]Present gage, located 65 feet downstream from Lincoln Way bridge, was installed in 1965. Prior to 1925, a non-recording gage was located 0.6 miles upstream from the current location, at a different datum. From March 11, 1925 through April 30, 1927, a non-recording gage was located 65 feet upstream from the present gage, at a datum approximately four feet higher than the present gage.

[b]No official gaging station maintained from May, 1927 - February, 1965.

[c]Property damage, evacuations, traffic flow interruptions and other flood impacts reported.

Table 1. (continued)

Date	Gage Height (feet)	Discharge (cubic feet/second)
Dec. 30, 1972	8.55[d]	1,590[e]
Jan. 18, 1973	10.80[d]	2,310[e]
Feb. 2, 1973	9.40[d]	2,540[e]
April 16, 1973	8.69	2,800
Oct. 12, 1973	8.64	2,750
May 16, 1974	7.21	2,080
May 18, 1974	8.14	2,450
June 9, 1974	7.93	2,400
June 19, 1974	8.02	2,440
June 22, 1974	8.95	2.900
June 26, 1975	9.79	3,430
June 27, 1975	14.00	11,300[c]
June 14, 1976	8.55	2,680
Aug. 8, 1977	7.09	2,070
Aug. 16, 1977	8.01	2,430
April 18,1978	7.11	2,060
Sept. 14, 1978	7.51	2,230
March 19,1979	11.81	5,300
Aug. 10, 1979	7.88	2,320
July 18, 1982	10.30	3,820
April 13, 1983	7.19	2,070
May 19, 1983	7.63	2,260
June 29, 1983	7.20	2,070
July 2, 1983	7.55	2,210
July 4, 1983	7.52	2,190
May 29, 1984	8.21	2,500
June 13, 1984	12.97	7,180
June 17, 1984	12.77	6,820

[d]Effected by ice.

[e]Estimated.

Mississippi River in southeastern Iowa.

In their analysis of the 1975 flood, Lara and Heinitz point out that this event seems to have been caused by an "ideally tuned" sequence of hydrologic events that amplified the basin response. The flood occurred at the end of an unusually wet June in which 11 to 13 inches of precipitation occurred in the Squaw basin. As a result, the daily average flow in Squaw Creek was abnormally high, exceeding 1100 cubic feet per second. In addition to this, very heavy localized rainfall of three to four inches occurred in the northern part of the basin during the early morning of June 25th. This caused a gradual rise to nearly bankfull flow at Ames. Finally, during the evening of June 26th, moderate amounts of rain fell during a thunderstorm that began over the headwaters of the basin and moved along a southeasterly path directly along the main channel of Squaw Creek.

Approximately 12 hours later, on the morning of June 27th, Squaw Creek crested at seven feet above flood stage. The peak flow of 11,300 cubic feet per second was approximately 1.6 times greater than the estimated 100-year flood.

Property damage in excess of $1,000,000 (U.S. Dept. of the Interior 1976) was caused by the 1975 flood, and one person drowned while wading in a flood-swollen tributary to Squaw Creek.

The greatest property damage occurred in the floodway corridor extending from North Sixth Street, on the Iowa State University campus, to South Duff Avenue. Near the north end of this corridor a dormitory parking lot, containing an estimated 400 vehicles, was flooded for several hours according to the Ames Daily Tribune ("Worst Flood Here Since 1918" 1975).

Slightly further downstream, two buildings in the Iowa State Center complex were completely surrounded by floodwater. Mechanical equipment and building materials housed in the lower level of the Scheman Continuing Education Center, then under construction, were extensively damaged. The Hilton Coliseum, just to the east of the Scheman building, was extensively flood-proofed during its construction and little damage was caused to this structure by direct entry of floodwater. Backflooding through the sanitary sewer system, however, did cause minor flooding in locker rooms and on the main floor in the lower level of this building (Dougal 1975).

Further to the south and east, Squaw Creek flows along the southern edge of a residential area. Despite efforts by Ames police to warn the owners, numerous automobiles in the parking lot of one apartment complex were damaged by rapidly rising flood water. Two homes also suffered severe damage when basement walls collapsed under excessive pressure from saturated soil. A nursing home in this same area was also extensively damaged when flood waters burst through windows rapidly filling the basement level.

In terms of dollar value, some of the greatest damage occurred in a commercial area along the west side of South Duff Avenue. A bowling alley, restaurant, movie theater, new car dealership, millwork, and offices for a well drilling firm and a chemical manufacturing company, were located in this area at the time of the 1975 flood. This commercial zone has continued to develop since that time and plans are now under way for additional development of a large tract of land on the south side of Squaw Creek.

Many businesses in the South Duff Area sustained considerable property and inventory damage during the 1975 flood. Last minute flood fighting efforts prevented some damage, however. Several firms reported that important records were preserved by removing lower drawers from filing cabinets and stacking them on top of the cabinets. Easily moved inventory items were saved by similar actions. Had there been more advanced warning, sandbagging and other emergency measures could have undoubtedly reduced flood damage even more (Dougal 1975).

Since the record flood of 1975, Squaw Creek has exceeded flood stage 16 times. The most serious of these occurred in 1984 when the flood crest fell only one foot short of that in

1975. While it seems doubtful that flooding along Squaw Creek is occurring significantly more often than it has in the past, it is clear that the potential for increased property damage from flooding grows each year as development intensifies in the floodplain.

Past Flood Control and Management Activities

Several courses of action are open to communities that experience flooding. In the early part of this century, considerable emphasis was placed on development of flood control structures. Reservoirs, levees, channel straightening, and other structural measures were commonly used to reduce or confine flood flows. During the thirty-year period ending in 1966, more than seven billion dollars was spent on flood control works in the United States (Peterson 1969, p. 158).

Following this approach to flood control, large-scale projects were proposed for Squaw Creek and the Skunk River in the late 1940s. Two reservoirs were proposed, one on Squaw Creek west of the town of Gilbert, and the other on the Skunk River a few miles upstream from Ames. The estimated combined cost in 1950 for these two reservoirs was approximately 8.7 million dollars. Strong opposition from local landowners prevented further action on these projects at that time.

In a later study of potential flood control measures in the Skunk River basin published in 1971, both reservoir sites were again studied. This time, only the Gilbert Reservoir on Squaw Creek was determined to be economically justified on the basis of flood control benefits. The estimated cost of the Gilbert Reservoir in 1970 was approximately 8 million dollars. Once again, strong local opposition to the project was the major factor in abandoning the project (U.S. Dept of Defense 1971).

By the 1960s, national research began to show that, despite massive investments in flood control structures, annual flood damages in the United States had grown to more than one billion dollars, more than twice the losses registered 30 years earlier. This disturbing trend showed that the rate of unregulated building in flood-prone areas was simply outstripping governmental efforts to protect these unwise developments. Unfortunately, this trend has continued with annual average flood losses in the United States reaching three billion dollars and continuing to rise at the end of the 1970s (U.S. Water Resources Council 1979).

Clearly, despite the risks involved, flood-plain development continues. The Federal Emergency Management Agency reports that nearly 20,000 of the 34,000 communities in the United States contain flood hazard areas (U.S. Dept. of Agriculture 1987, p. 8-2). This is due, in part, to poor public understanding of flood hazards, and a tendency to quickly forget the impacts of infrequent natural disasters.

Furthermore, some government programs, despite their intent, seem to encourage occupation of flood-prone lands. In their 1979 report to the President on floodplain management, the U.S. Water Resources Council stated:

> "The customary sequence of events generally continues to be (1) flooding, (2) flood losses, (3) disaster relief,(4) flood control projects attempting to modify the flood potential through provisions for storing, accelerating, blocking or diverting flood waters, (5) renewed encroachment and development onto the floodplain and upstream watershed, (6) flooding, (7) flood losses, (8) disaster relief, (9) more projects, (10) more encroachment and development, ad infinitum."

In light of continuing floodplain development and the realization that structural measures alone cannot stem the growth in annual flood damages, officials of flood-prone communities have come to recognize that a strong flood hazard mitigation program must incorporate both structural and non-structural elements. Public education programs, floodplain zoning, flood prediction and warning systems, and development and implementation of emergency action plans are just a few of many non-structural measures that can reduce flood hazards.

In his report on techniques for developing a comprehensive program for floodplain management, Dougal (1969, p. 53) outlined an eight-point community program, involving both structural and institutional measures, to minimize flood hazards. The key features are as follows:

1. Recognition of the flood hazard by the community;

2. Implement and maintain a flood forecasting and warning system;

3. Develop detailed emergency operating procedures for flood fighting;

4. Outline a program for adjustments in structures and occupancy in flood hazard areas;

5. Implement floodplain regulations;

6. Carry out technical, socioeconomic, and legal-institutional studies for optimal future utilization of the floodplain;

7. Construct engineering works that are part of the comprehensive floodplain management plan; and

8. Operate and maintain the comprehensive floodplain management plan.

To date, Iowa State University and the city of Ames have focused their flood protection efforts on measures in categories 4, 5, 6, and 7 in the plan outlined above.

In 1956 Wells examined the flood potential of the Squaw and Skunk basins by studying five of the largest storms of record in the Upper Midwest. Using rules developed by the U.S. Weather Bureau, Wells transposed these storms over the Squaw and Upper Skunk basins and estimated the flood hydrographs that they would cause. He concluded that the largest floods of record in these basins were minor when compared with floods that could result from record storms that have occurred in other parts of the Midwest.

In the early 1960s Vawter (1963) developed water surface profiles along the Squaw Creek floodplain in Ames for floods of various magnitudes.

Efforts to map flood-prone areas and regulate their use began in the mid 1960s with a study of the Squaw Creek and Skunk River floodplains by the Rock Island Corps of Engineers (U.S. Army Engineer District 1966). This study, which was entered into cooperatively by the city of Ames, Iowa State University, and the Iowa Natural Resources Council, set the stage for development of floodplain zoning ordinances in the 1970s. A complete analysis of current land use in the floodplain and recommendations for future regulation and use of the floodplain within the city limits was published in 1975 (City of Ames).

Adjustments to structures occupying the floodplain can be seen in dormitories and buildings in the Iowa State Center complex, both of which were built on the western edge of the Squaw Creek floodplain in the past two decades. Earthen berms, elevated first floor entrances, and extensive under-drainage systems are examples of flood damage reduction features incorporated in the design of these structures.

Major structural projects were also constructed to channel flood flows away from high value property adjacent to Squaw Creek. Elwood Drive, a major four-lane thoroughfare which provides access to the Iowa State Center, was constructed on an elevated grade to form a levee between Squaw Creek and the Center. In addition, a levee was constructed along the lower portion of College Creek, a tributary to Squaw Creek that flows through the Iowa State campus.

Following the floods of 1975 and 1984, city and university officials became concerned that a potentially useful element was missing from their flood damage control program. This element was a flood forecasting and warning system. Experience during the 1975 flood showed considerable property damage can be averted if advanced flood warning is given. In response to this, city and university officials contacted Dr. T.A. Austin, Director of the Iowa State Water Resources Research Institute, and requested that he investigate the feasibility of a flood prediction and warning program. The remainder of this dissertation outlines work done to date on this project.

STUDY OBJECTIVES AND CONSTRAINTS

To help facilitate development of an effective and low cost system for predicting flooding along Squaw Creek, a Squaw basin flood study was undertaken with three major objectives.

The first is to identify key hydrologic parameters necessary to predict basin response to heavy rainfall. Since each additional forecasting parameter increases the costs of operating a flood forecasting and warning program, emphasis was placed on minimizing data input requirements.

The second objective is to develop a simple flood prediction procedure or model, based on the key parameters previously identified, which could be used by city officials to help determine when to issue a flood alert.

The third major objective of this study is development of recommendations for implementation of a flood prediction and warning program. These recommendations are to cover input data requirements, methods and frequency of data collection, personnel requirements, and equipment needs.

As is often the case in engineering projects, the challenge lies not only in application of theory, but in meeting client-imposed constraints on the scope of the development effort and the form of the final product.

During initial discussions of this project, city officials made it clear that operational simplicity and low cost were key constraints. Operational simplicity was particularly important since local law enforcement officials, not trained hydrologists or engineers, would be using the flood prediction procedure.

Funding for development of the flood prediction program was also limited. Since there was no money for field data collection, secondary data sources had to be relied on. Soil surveys, topographic maps, rainfall and streamflow records, and stream valley cross sections at bridge and culvert sites, were used to estimate essential basin parameters needed to predict the rainfall/runoff response.

Finally, maintenance and operating costs of the proposed flood prediction and warning program were to be kept to a minimum. A predictive model requiring large quantities of input data or costly large-scale computer facilities was not desired. Similarly, city officials felt that sophisticated automated data acquisition networks would be difficult to justify since, despite the fact that the threat of flooding occurs often, the frequency of damaging floods in Ames is not high.

REVIEW OF LITERATURE

Much has been written about flood prediction and warning systems in the past 20 years. To summarize information most pertinent to this study, representative examples from the literature will be broken into two general categories. These are:

1. Reports on operational flood forecasting programs, including their organization, forecasting procedures, equipment, cost, and effectiveness; and

2. Investigations of new developments in flood prediction which have not yet been widely applied in practice.

Operational Flood Forecasting Programs

The scope of river forecasting operations varies widely, from large regional or national programs that monitor major river systems, to small community self-help projects that focus primarily on flash flood predictions for local streams.

Large-Scale River Forecast Operations

Programs in the U.S. According to Wood (1980, p. 245), the United States probably has the most extensive operational river forecast system in the world. Flood forecasts for major river systems are prepared by 13 regional River Forecast Centers operated by the National Weather Service (U.S. Dept. of Commerce 1985). River Forecast Centers prepare flood forecasts and warnings for roughly

3,000 communities throughout the nation.

Observations from river and rainfall gaging stations, radar imagery, and visible and infrared satellite imagery, are primary sources of meteorologic and hydrologic input data. These data, and the quantitative precipitation forecasts developed from them, are used to forecast flood crest, time of crest, and duration of flooding at various locations. Flood forecasts are released to the public through National Weather Service Offices located in all 50 states and through state and local disaster management agencies.

River forecasting procedures used by the National Weather Service have changed substantially in the past 20 years. Weather Bureau Technical Memorandum WBTM HYDRO 9, entitled "Elements of River Forecasting (Revised)" (U.S. Dept. of Commerce 1969), summarizes forecasting procedures in use at the time it was published.

This document, which was reprinted in 1975, explains procedures for development and use of a graphical rainfall/runoff model based on coaxial correlation methods as originally described by Linsley, Kohler and Paulhus in 1949. Using this procedure, storm runoff is predicted based on soil moisture, week of the year, storm duration, and storm precipitation. Unit hydrograph theory, and a graphical reservoir routing procedure using lag and storage coefficients, were used to transform predicted surface runoff into a flood hydrograph at a particular forecasting point in a basin.

A distinctive feature of this model, one for which it is named, is the procedure used to quantify basin-wide soil moisture. This is accomplished through use of an Antecedent Precipitation Index (API) which reflects the amount and time of occurrence of precipitation occurring in the basin prior to the particular storm event for which a runoff estimate is desired. According to HYDRO 9, the API is theoretically the sum of an infinite series:

$$API = b_1P_1 + b_2P_2 + \ldots\ldots + b_iP_i \qquad (1)$$

where:

P_i = precipitation, in inches, occurring on the ith day prior to the storm event under consideration;

b_i = series of constants where $b_i < b_2 < b_1 < 1.0$

To simplify daily computation of the API, it is generally calculated as a fraction of the API at the beginning of the previous day plus any precipitation which occurred during that day. The defining equation then becomes:

$$API_t = kAPI_{t-1} + P_{t-1} \qquad (2)$$

where:

API_t	=	API at beginning of day t
P	=	precipitation during day t (24 hour precipitation in inches)
k	=	a constant ranging from 0.85 to 0.95

Although digital computers were available in the late 1960s, the Weather Bureau literature says little about their use in river forecasting at that time. HYDRO 9 briefly alludes to use of digital computers for automating the development of rainfall/runoff correlations, but most of the procedures it describes are manual operations. No specific mention is made regarding availability or use of forecasting software.

At nearly the same time that HYDRO 9 (revised) was published in 1969, however, Sittner, Schauss, and Munro (1969) reported development of an extended API-type computer model capable of continuous hydrograph synthesis. This model included a new parameter, called the Retention Index, which was used to adjust the API to account for increased runoff caused by changes in interception, depression, and soil moisture storage during extended rainfall.

In addition, a groundwater flow component was incorporated so that continuous river forecasts could be made. This simple model successfully simulated continuous historical streamflow sequences of up to ten years in length.

Because of its relative simplicity and good performance, this model became a standard of comparison for new river forecasting procedures developed by the National Weather Service in the 1970s.

In the early 1970s, the National Weather Service initiated a major change in river forecasting procedures as it moved from empirical models to theoretically based hydrologic relationships. Curtis and Smith (1980) list four reasons for this shift to theoretical forecasting models:

1. Accurate mathematical representation of a catchment enhances the probability of adequately predicting future events of magnitudes not experienced in the past;

2. Parameters based on conceptual considerations can be altered to reflect changes in physical characteristics of a watershed;

3. A conceptual model may be extended to applications other than steamflow simulation, such as modeling of pollutant movement: and

4. A physically based model is an effective tool for future research.

Under the direction of the Hydrologic Research Laboratory, three computer-based continuous streamflow models were tested. These were the Streamflow Synthesis and Reservoir Regulation Model, developed by the U.S. Army Corps of Engineers, the Sacramento River Forecast Center Hydrologic Model, and a modified form of the Stanford IV Model based on the work of Crawford and Linsley (U.S. Dept. of Commerce

1972).

Following extensive evaluation, the modified Stanford IV Model was selected for further development and use in the river forecast software package which became known as the National Weather Service River Forecast System. In addition to the Stanford Model, this package contained subroutines for estimating missing data, temporal distribution of cumulative precipitation data, and calculation of mean areal precipitation.

Since the early 1970s several important modifications have been made to the National Weather Service River Forecast System (Curtis and Smith 1980, p. 308).

1. The Stanford soil moisture accounting model has been replaced by a model developed by the National Weather Service River Forecast Center at Sacramento, California;

2. A snow accumulation and ablation model has been added;

3. A dynamic river routing model has been added; and

4. Data management capabilities have been greatly expanded.

Operational use of the National Weather Service River Forecast Model began in the late 1970s when it was installed at a central computing facility in Maryland. River Forecast Centers throughout the country access the model through remote terminals. Curtis and Smith (1980) estimate that five to ten years will be needed for nationwide implementation by all river forecast centers.

The slow pace of adoption is due, in part, to the large data requirements of the model. Approximately 10 years of meteorological and streamflow data are recommended for satisfactory model calibration.

The substantial number of model parameters also complicates the calibration process. To assist forecasters with calibration, a Direct Search Optimization subroutine, as described by Munro (U.S. Dept. of Commerce 1971), is included in the model. A Pattern Search technique is used to reduce the time needed to identify optimal combinations of parameters. The pattern search procedure is an iterative one that begins with small sequential trial and error adjustments to each model parameter. Adjustments which significantly improve model performance are identified and subsequent adjustments to these key parameters are systematically increased in size until the objective function no longer shows improvement. Parameter adjustments that do not improve the objective function are phased out.

Programs in Other Nations In terms of its organization, the Australian approach to flood forecasting is similar to that in the United States. Overall responsibility for flood forecasting lies with the Bureau of Meteorology of the Commonwealth Government Department of Science. Regional forecast centers located in the capitol cities of four eastern states provide qualitative and quantitative flood forecasts in as many as 25 basins in some states.

In their report to the International Symposium on Logistics and Benefits of Using Mathematical Models of Hydrologic and Water Resource Systems, held in 1978, Hall and Elliott (1981) outlined a new plan for automating and improving the Australian forecasting service. The new system, called the Automated Regional Operations System, consists of mini-computers located in regional forecast centers which are linked with a larger computer at a central forecast office. The main purpose of the mini-computers is to collect and relay field data to the central computer and to display charts, maps, flood alerts, and other forecast documents from the central office.

Because of the tremendous variety of hydrologic conditions and the widely varying availability of real-time field observations in Australia, no single river forecast model will handle all conditions. For this reason, the Australian system incorporates several hydrologic models ranging from those that are complex and theoretically based, to simple nomographic relationships.

In 1981 Bergstrom reported that river forecasting in Sweden was underway in 10 major basins. Seasonal forecasts to optimize hydroelectric power generation is the primary goal, but predicting spring snowmelt flooding is also an important forecasting activity in some basins.

Due to a limited meteorological data base, a relatively simple hydrologic model was developed by the Swedish Meteorological and Hydrological Institute. The model is run on a daily basis using mean air temperature, precipitation, and monthly standard values of potential evaporation as primary inputs. The model has 13 empirical coefficients which have to be estimated during calibration. Experience has shown that many of these coefficients vary only slightly from basin to basin, however.

The probability of extreme flooding during spring snowmelt is estimated using early spring measures of snowpack depth and soil moisture conditions. The model is then run using many historically recorded sequences of spring temperature and rainfall to generate a range of spring river flow scenarios. Statistical analysis of the model output provides a basis for estimating seasonal flood risk.

Flow forecasting in the Dee River Basin in Wales reflects practices used on Great Britain (Cole 1980). As in the United States, operational British forecasting models are largely deterministic.

The Dee River System includes four multi-purpose reservoirs that are operated for water supply, flood control, and river regulation. These reservoirs are located in basin headwater regions and control a relatively small portion of total basin runoff. To avoid causing unnecessary downstream flooding their operation must be carefully coordinated with river flow predictions for the uncontrolled portion of the basin.

A relatively simple rainfall-runoff model predicts outflow based on the amount of precipitation stored on the basin. Telemetering rain gages provide real-time data on precipitation in each subbasin. Weather radar provides rainfall intensity data that is used to estimate rainfall in subbasins that do not have telemetering gages. River routing is based on a modification of the Muskingham method which uses variable routing parameters that are dependent on stream discharge.

Small-Scale River Forecast Operations

National Weather Service flood stage forecasts are routinely issued for about 3,000 locations across the nation. Most of these are for large cities located along major streams. But there are an estimated 20,000 additional locations subject to flooding where insufficient staff and data are available for detailed National Weather Service flood forecasts. In these areas Weather Service flood forecasting is limited to headwater advisories, urban flooding statements, and other generalized warnings based on radar, satellite imagery, regional streamflow data, and scattered rainfall reports. These generalized warnings often refer to one or more counties, and flooding is often in progress when they are issued (U.S. National Weather Service 1985).

Several factors prevent national and regional programs from providing effective local flood forecasts in small basins. The most severe problem is inadequate precipitation data. The Weather Service precipitation gage network averages about one gage every 300 square miles, but many floods are caused by localized severe storms affecting less than 50 square miles. Infrequent rainfall reports also limit flood forecasting accuracy. In many cases Weather Service gages are read only once a day. At best, rainfall reports are obtained only once every six hours unless gages are automated. For small basins with short response times, six-hour rainfall reports do not allow sufficient time for preparation of flood forecasts, dissemination of warnings, and implementation of emergency measures before flooding begins.

Lack of detailed physical and hydrologic data for small basins also makes it difficult to calibrate and verify hydrologic models for these regions.

To cope with the need for better data collection and forecasting in small basins, a broad range of flood forecasting programs have been implemented at the community and county level. Some, like the San Diego County program, employ sophisticated automated data collection and analysis systems. Others rely on volunteer rainfall observers and a few simple charts or graphs that relate streamflow to rainfall in a particular basin.

According to the Bulletin of the American Meteorological Society ("First Countywide Real-Time" 1982), the San Diego County project is the nation's first county-wide, real-time flood warning system. It consists of 17 rain gages, 20 stream gages, and 4 repeater stations that monitor 11 reservoirs and 9 major streams in a 4300 square mile region.

The data collection network is a "transmit-only" system rather than an "interrogated system". This means that precipitation and streamflow gages continuously monitor and report changes rather than responding to queries from a central data storage and processing facility. As a result, field stations need only to operate in a transmit mode. This cuts cost and reduces the risks of equipment failure since field receivers at each station have been eliminated.

Data analysis and forecasting are done by a computer using river forecast models provided by the National Weather Service. The forecast is automatically updated every 12 minutes. An audible alarm is sounded if flood stage threshold levels are exceeded or are predicted to do so.

The San Diego County project, which cost $270,000, is part of a relatively new National Weather Service program called ALERT (Automated Local Evaluation in Real Time). As of late 1982, a total of 40 counties located in California, Colorado, Arizona, Texas, Minnesota, Connecticut, and New York, had installed, or were planning to install, ALERT systems.

Curtis and Greechan (1984) have described the ALERT system which serves Westchester County, a heavily populated 450 square mile area north of New York City. This heavily developed region has suffered over 44 million dollars in flood damage since 1974. Small heavily-developed stream basins with short response times made a real-time forecasting system essential for this region where transportation arteries that carry more than 400,000 motorists per day can be rapidly flooded.

The Westchester forecasting system consists of 11 precipitation gages, two temperature sensors, six stream gages, one radio repeater station, two base stations, and a portable micro computer. The hydrologic model used is the Sacramento Soil Moisture Accounting Model which was developed by the National Weather Service.

The authors emphasize the operational flexibility and reliability of the battery powered rain gages that have been designed for ALERT systems. The tipping bucket mechanism transmits a 250 millisecond radio signal each time one millimeter of precipitation accumulates. Since the reporting frequency is tied directly to storm intensity, data on both storm intensity and accumulated precipitation are readily available.

Because radio transmissions are only about one quarter of a second in length, electrical power requirements are small. In a climate with 1000 millimeters of annual precipitation, total annual radio transmission time, including two daily test transmissions, is less than eight minutes. As a result, these gages can operate from remote locations for over a year without a battery change.

The Integrated Flood Observation and Warning System (IFLOWS) is another special flash flood program sponsored by the National Weather Service. It is similar in concept to the ALERT projects, but somewhat larger in scale. IFLOWS has been installed in a 95-county area located in Virginia, West Virginia, Tennessee, Kentucky, and Pennsylvania. Approximately 600 solar-recharged battery-powered rain gages have been installed in this large Appalachian Mountain region. These transmit real-time precipitation data, by line-of-sight VHF radio, to computers in weather forecast offices in the five-state area. Using telemetered precipitation and streamflow data, flash flood alerts can be rapidly developed and disseminated to the affected areas (Most 1984).

A combined flood forecasting and reservoir operation model was developed by Eggert, Huang, and Ballantine (1984) for the 3100 square mile Upper Pearl River basin in central Mississippi. The basin is modeled as a system of 133 planar watershed units linked by channel segments. Overland, channel, and subsurface flow are all modeled using the kinematic wave formulation.

Infiltration is assumed to be approximately equal to the saturated soil conductivity, and deep percolation is computed as a percentage of infiltrated volume. The remaining infiltration is routed horizontally through the soil as interflow. This model has been successfully installed and calibrated for interactive use in reservoir operations on the Pearl River.

At the opposite end of the technology and cost spectrum are two very simple "self-help" flood forecasting and warning programs developed by the National Weather Service for use in small basins that are not covered by river forecast center operations (U.S. Dept. of Commerce 1979, 1980).

The simplest of these is an upstream flow monitoring system. In its most elementary form this consists of a staff gage installed upstream of a flood-prone community. During and following heavy precipitation, the upstream gage is monitored by a local observer. Peak stage information is relayed to the downstream community where crest-lag charts are used to predict peak stage and time of arrival at the downstream damage center. Both radio or telephone telemetry can be used to monitor the upstream gage thereby eliminating the inconvenience of stationing an observer at the gage. In some systems an audible or visual alarm located at a law enforcement or disaster services office is triggered when the stage at the upstream gage reaches a preset elevation.

An effective upstream monitoring and warning program requires a suitable gaging site located far enough upstream to afford several hours of warning time to the flood-prone community. In addition, there must be no major tributaries contributing flow between the gage and the downstream community as this makes it difficult to define a consistent crest-lag relationship (Linsley, Kohler, Paulhus 1982, p. 282).

Development of a crest-lag relationship is based on data from past flood events. Corresponding peak stages at the upstream gage site and a damage center are plotted on cartesian coordinates. With this plot and knowledge of the approximate flood wave travel time between the gage and the community, rapid projections of peak stage and its time of arrival can be made.

A more sophisticated version of crest-lag forecasting, using average rainfall intensity as an additional input parameter, is described by Mimikou, Skaltsas, and Methanis (1984). In this particular case, comparison of peak stage at the upstream location and at the forecasting point showed two distinct basin response curves that are dependent on average rainfall intensity measured at a location midway between the two stream gaging stations. By using the rainfall intensity data to differentiate varying basin response, the authors claim improved flood forecasting abilities using crest-lag methods.

When a flood damage center is located near the headwaters of a basin, an upstream monitoring system may not provide sufficient warning time prior to onset of flooding. Longsdorf (U.S. National Weather Service 1985) reports that in both 1978 and 1982 upstream warning stations at Austin, Minnesota failed to give adequate warning of floods caused by heavy localized rainfall in the basin headwaters. In situations such as these, where basin response is rapid, a locally operated rainfall reporting network and flood forecasting program is recommended by the National Weather Service.

This type of self-help forecasting program is generally implemented as a cooperative project between the National Weather Service and a local unit of government. The Weather Service designs the data collection program, provides training for local personnel, and prepares a series of simple flood warning tables that make it possible for community officials to develop emergency flood forecasts based on local rainfall data. The local unit of government maintains the precipitation gage network, prepares a flood emergency action plan, and disseminates flood warnings to affected areas.

Fox and Hurst (1976, 1980) describe procedures used by the S.E. River Forecast Center in development of flood warning tables for small communities in their region. A simple rainfall-runoff relationship similar to that in Figure 2 is derived from rainfall and runoff data for past flood events. It is assumed that no significant runoff occurs until the soil moisture deficiency is satisfied. Soil moisture deficiency is defined by equation 3:

$$DE = DB - R + E \tag{3}$$

where:

DE = soil moisture deficiency at the end of a computational period;

DB = soil moisture deficiency at the beginning of a computational period;

R = Rainfall (or snowmelt) during the computational period;

E = Evapotranspiration during the computational period

A negative value for DE indicates rainfall in excess of that needed to bring the soil to field capacity and is the rainfall excess shown in the storm runoff relation in Figure 2.

Evapotranspiration is estimated using ratios of actual evapotranspiration to potential evapotranspiration that have been associated with various levels of soil moisture deficiency in the basin.

Once a rainfall-runoff relationship is developed, unit hydrographs are used to obtain basin outflow hydrographs for various amounts of runoff. Flood routing procedures are then applied to estimate flood discharge at downstream damage centers where stage-discharge tables indicate flood crest elevations.

To simplify the flood forecasting process as much as possible, the flood crests predicted for various

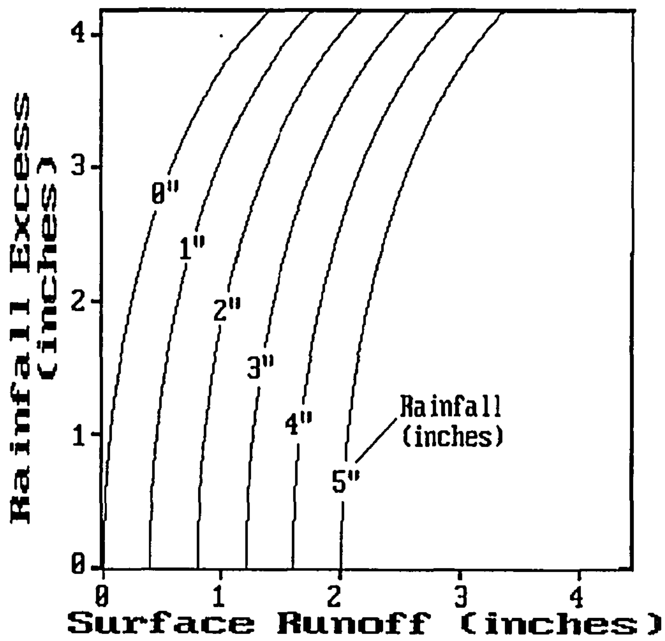

Figure 2. Relationship between rainfall excess, rainfall, and runoff for Peachtree Creek at Atlanta, GA. (adapted from Fox and Hurst 1980)

combinations of rainfall, storm duration, and soil moisture deficiency are arranged in tabular form like Table 2.

Using basin rainfall data supplied by local observers, and the soil moisture index which is transmitted daily by the nearest river forecast center, a local forecaster simply selects the appropriate flood warning table for the indicated soil moisture index and reads the estimated crest stage from the appropriate row and column.

According to Fox and Hurst, experience with flood warning tables for 90 floods in the Southeastern United States has shown an average forecast error of 0.76 feet. Insufficient rainfall data were listed as the most common source of forecast error.

Planning and Evaluation of Flood Warning Programs

Equipment Selection Recent advances in micro-electronics have added many new products to the list that flood warning system planners can choose from. The current literature, however, seems to offer little general guidance regarding availability and selection of system components. This may be due, in part, to the wide diversity of forecasting needs or to the rapid growth of available technology.

The most comprehensive catalog of system components appears to be a document published by the U.S. Department of

Table 2. Portion of a flood warning table developed by the S.E. River Forecast Center (adapted from Fox and Hurst 1980)

Flood Warning Table
for
Cedar Creek at Cedartown, GA.
Flood Stage 12.0 Ft.
(for soil moisture deficiency 0.0 - 0.5 inches)

PREDICTED CREST STAGE (FEET)[a]

Rainfall in last 2 hours	Duration of storm in hours	Rainfall in inches over the basin in past 24 hours						
		1.0	1.5	2.0	2.5	3.0	4.0	5.0
	4	8	10	13[b]	15[b]	17[b]	22[b]	25[b]
0.5	6	8	10	12[b]	15[b]	16[b]	20[b]	23[b]
	12	7	10	11[b]	13[b]	15[b]	176[b]	20[b]
	24	7	9	10	12[b]	13[b]	15[b]	16[b]
	4	8[c]	11	13	15	16[b]	21[b]	25[b]
1.0	6	8[c]	11	13	14	16[b]	20[b]	23[b]
	12	8[c]	10	12	14	15	17[b]	20[b]
	24	8[c]	10	12	13	14	15[b]	17[b]
	4			13[c]	15	17	20[b]	25[b]
2.0	6			13[c]	15	17	20[b]	24[b]
	12			13[c]	15	16	19	22[b]
	24			13[c]	15	16	18	20
	4					18[c]	21	24[b]
3.0	6					17[c]	21	24[b]
	12					17[c]	21	23
	24					17[c]	21	23
	4						22[c]	24
4.0	6						22[c]	24
	12						22[c]	24
	24						22[c]	24

[a]The crest stage normally occurs about 6 to 7 hours after the end of heavy rainfall.

[b]Due to previous heavy rainfall or more rapid response in extreme floods, the crest stage should occur at least 2 hours earlier than normal and may have already occurred.

[c]Rainfall duration for these values is 2 hours

Commerce (1981) entitled Equipment for Flood and Flash Flood Warning Systems. This publication summarizes specifications for water level detectors, flow measuring devices, precipitation gages, telemetry equipment, warning sirens, and many other components that are offered by approximately 80 different manufacturers.

Increased availability of low-cost microcomputers and radio telemetry equipment has played an important role in automation of flood forecasting and warning systems. Although the initial costs of fully automated systems are higher than for manual or semi-automated systems, the lifetime costs of these systems has been shown to be competitive because they are less labor intensive to operate. In their comparison of equivalent semi-automated and micro-computer controlled flood warning systems in New York, Burnash and Bartfield (1980) show the micro-computer controlled system to have lower total capital and operating costs over a ten year period.

Organization and Communication An effective flood warning program involves data collection and analysis, preparation of streamflow forecasts, evaluation of flood stage at various potential damage centers, dissemination of flood warnings in threatened locations, and implementation of evacuation plans and other emergency flood damage mitigation measures. Clearly, even small local programs can be quite complex, and careful planning, evaluation, and fine tuning are needed to make a program workable and effective.

In their evaluation of National Weather Service flash flood operations, Belville, Crouch, and Hollis (1980) observed that this program has demonstrated only limited success in dealing with flash flood problems. They suggest five ways in which Weather Service Forecast Offices could improve flash flood operations. Included are:

1. Better forecasting of excess precipitation through use of the National Meteorologic Center Excessive Rainfall Outlook, and development of local forecasting tools and procedures which identify local atmospheric parameters that indicate heavy rainfall potential;

2. Improved detection and measurement of excess precipitation by expanding the cooperative observer network and through more extensive use of manually digitized radar and satellite imagery to obtain supplemental rainfall estimates;

3. Increased communication and coordination between Weather Service Forecast Offices, Weather Service Offices, and other elements of the National Oceanic and Atmospheric Administration;

4. More extensive training for forecasters; and

5. Improved organization of the flash flood operations area at each weather forecast station.

In his review of Flash Flood Preparedness Procedures, Hutcheon (1980) emphasized the need for a "storm coordinator" at each weather service office. Noting that most offices are normally staffed for "fair weather" conditions, it was suggested that an on-call storm coordinator be assigned to serve as a liaison with state and local governments, the mass media, and other weather service offices.

The details of organizing and operating a local flash flood warning system are summarized by Braatz and Sisk (1980). Special emphasis is placed on personnel needs at the community level and on the importance of establishing a Flash Flood Coordinator who is sufficiently trained to manage a local emergency action plan and to serve as a liaison between the National Weather Service, Civil Defense officials, rainfall observers, and other elements of a local flood response team.

A recommended organizational structure (Figure 3) for the operational elements of a local flood warning unit is outlined and legal responsibilities of the local unit and the National Weather Service are discussed. The authors stress that, by law, the National Weather Service is responsible for issuing flash flood watches and warnings and that a memorandum of understanding between the unit of government in charge of the local flood warning program and the National Weather Service is essential to clarify the responsibilities of each party in a local cooperative flood warning program.

<u>Local Concerns</u> In a study of reasons cited by local officials for <u>not</u> implementing local flood warning programs, Owen (1980) lists ten common concerns (Table 3). Based on a survey of directors of state emergency service agencies, Owen concluded that there is widespread misunderstanding of local

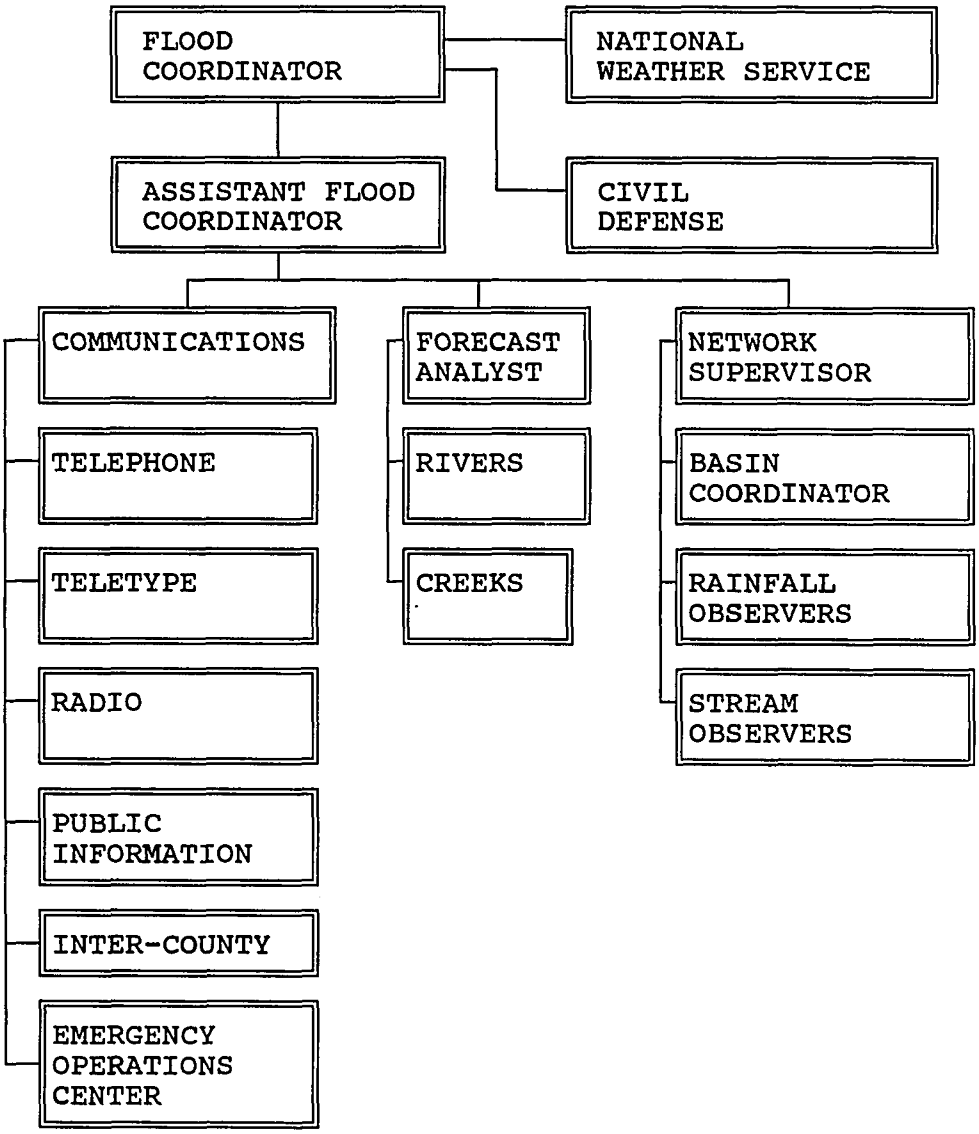

Figure 3. Organizational chart for a local flood warning unit (Braatz and Sisk 1980)

Table 3. Reasons cited by local officials for not implementing a flood warning program (Owen 1980)

Reason Expressed	Percent of Time Mentioned
Financial Concerns	23
Misconceptions of Flood Problem	18
Lack of Understanding of Warning System	14
Legal Concerns	10
Apathy	9
Perception of Responsibility for Warning	8
Local Organizational Arrangements	7
Political or Personal Concerns	5
Lack of Technical Capability	3
Miscellaneous	3

flood warning programs and that many officials do not recognize the severity of the flood hazards in their communities.

One of the concerns most commonly voiced by local officials is for the possible legal liabilities of operating a local flood warning program. Many officials feared accountability for inaccurate flood forecasts that cause unnecessary evacuations and disruptions of commerce, or that fail to forecast and warn the public of an impending flood. Although further nationwide research on the legal liability issue was recommended, Owen pointed out that none of the several hundred local flood warning programs in operation throughout the country had reported any legal problems to the National Weather Service as of the time of his study in 1978.

Human Response to Flood Warnings No matter how accurate and timely a flood forecast is, little benefit is derived unless a warning is effectively disseminated to the public and appropriate emergency measures are taken by those in flood hazard areas.

Several writers have dealt with improving human response to flood warnings. Mogil (1980) reviewed the role of the mass media during weather emergencies. He concluded that forecasting operations must make an effort to understand media operations in order to work effectively with the media and that deliberate measures must be taken by forecasting agencies to establish, test, and improve their linkage with the mass media.

Tamminga (1980) analyzed the response of Texas Hill Country residents to a devastating flood in 1978. Based on a survey of flood victims, she concluded that flood warning programs must incorporate more than one method of warning dissemination since telephone, electrical power system, and radio transmitter failures are common during storms. Secondly, people are more likely to take action if a warning is received via two different modes. Warning sirens alone are not always effective since they do not clearly indicate the exact nature of an emergency. Sirens supplemented by local radio and television broadcasts or by warnings issued via loudspeaker systems on law enforcement vehicles are much more effective since the nature of the emergency is more fully portrayed.

In their assessment of benefits of a flood warning system, Day et al. (1969) point out that local reactions to flooding and flood warnings are often correlated with the frequency of flooding. As a result, flood warning plans must vary from one community to another. In communities where flooding is common, it may take relatively little warning to cause the public to take appropriate emergency measures. Where flooding is infrequent, several concurrent warning modes may be necessary to obtain appropriate public response.

In their study of perception of natural hazards, Burton and Kates (1964) analyzed variability in perceptions of hazards among various social groups and the ways that this affects response to hazardous situations. Considerable differences in hazard perception are noted between scientific personnel, who are used to dealing with uncertainty, and the general public. To be effective in eliciting public response, flood warning programs must recognize locally held perceptions of flood hazards and utilize educational programs to overcome inaccurate perceptions that hinder implementation of appropriate emergency measures.

<u>Evaluation of Economic Benefit</u> Although most planners and engineers agree that flood warning programs are beneficial, placing an economic value on these benefits has

been quite difficult. The biggest problem is that flood warning programs, like all non-structural flood damage mitigation measures, require personal involvement or response by floodplain dwellers.

Given adequate lead time, considerable property damage can be averted by moving valuable items to high ground and by installation of temporary flood barriers. But if floodplain dwellers underreact or overreact to a warning, the actual economic benefits derived may be only a small percentage of the potential benefits. Predicting or quantifying the human response factor is difficult at best. For this reason most attempts to assess the economic value of flood warning programs must be based on some assumed level of human response.

Several authors have presented procedures for evaluation of flood warning system benefits. Day et al. (1969) developed stage-damage-warning time curves, like those shown in Figure 4. These curves represent estimated average damage incurred by various classes of property in a community as a function of flood depth and amount of advanced flood warning given to property owners. When this information is combined with stage-frequency data, annual community-wide damage for differing levels of flood warning and emergency preparedness can be calculated. By comparing estimated damages with and without a flood warning program, the economic benefits of

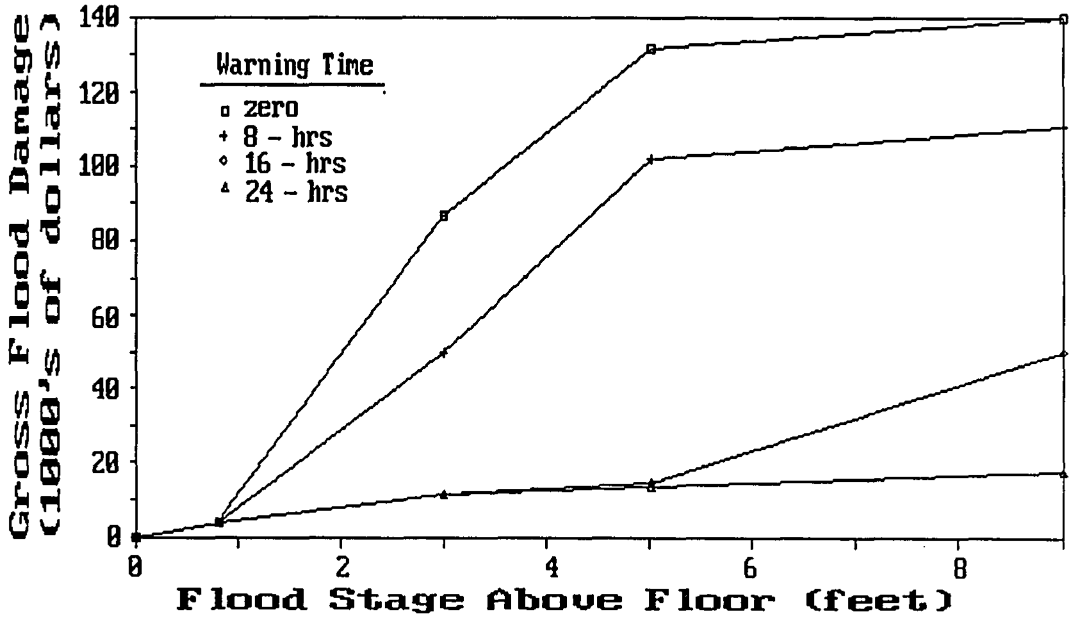

Figure 4. Example of stage-damage-warning time curves for a supermarket (Day et al. 1969)

flood warning systems can be quantified.

Using a similar but more data intensive approach, Day and Lee (1976) developed damage versus stage graphs for several categories of structures at four flood-prone locations along the Connecticut River. Damage reduction afforded by three different non-structural alternatives (partial evacuation of moveable items, complete evacuation of moveable items, and partial relocation of moveable items to storage areas at second story level) were evaluated to assess the benefits of various levels of flood warning. Results were generalized for several characteristic types of flood-plain developments and structures. These generalized relationships were then applied to similar floodplain developments throughout the river basin to estimate the basin-wide flood damage reduction potential of a flood warning system.

Recognizing that previous economic evaluations of flood warning programs were based on assumptions of perfect forecasts and optimal human response, Sniedovich and Davis (1977) developed a mathematical forecast-response model that recognizes uncertainties in a flood warning and emergency response program. Through application of decision theory, their model determines an optimal response strategy for floodplain dwellers assuming various levels of uncertainty in the flood forecast and the floodplain dweller's perception of the flood hazard. With sufficient data the model can be used to evaluate benefits associated with improvements in forecasting, warning dissemination, public awareness, and emergency response.

Design and evaluation of non-structural flood damage mitigation projects is a data intensive trial and error procedure. In an effort to facilitate this process the Hydrologic Engineering Center of the U.S. Army Corps of Engineers has recently developed several computer software packages (Ford 1981, Johnson and Davis 1984). These programs are designed to catalog and retrieve data on frequency of flooding at various depths, stage-damage relationships, property values, and feasible protection alternatives for each structure in a flood hazard area. These data can then be used to evaluate the potential benefits of proposed non-structural floodplain protection programs.

Recent Developments in Flood Forecasting

New data collection equipment, increased accessibility to computers, and development of real-time forecasting techniques have made new flood forecasting techniques feasible in the past ten years. Although much of this new technology is yet to be widely adopted for operational forecasting, some of it undoubtedly will become common practice as demand grows for more accurate and timely forecasts at many locations.

New Equipment

Advancements in earth satellite technology are rapidly changing the way hydrologic data are collected and communicated to forecasters. As noted earlier, the National Weather Service makes widespread use of satellite imagery for weather and river forecasting, but with the expanding availability of satellite services and data, smaller forecasting operations now also have access to this technology.

Yates and Anthony (1985) report that the U.S. Army Corps of Engineers has installed nearly 400 satellite data collection platforms throughout the Ohio River basin to facilitate river forecasting and operation of 77 reservoirs.

Field sensors collect hydrometeorological data and transmit this information hourly, via the data collection platforms, to the Geostationary Operational Environmental Satellite (GOES) system. The GOES system relays the field data to a computer at the Ohio River Division Control Center in Cincinnati for use in scheduling reservoir releases for optimal flood control and water management.

Land use data obtained through satellite remote sensing operations is also being used in hydrologic modeling. Alexander and Rao (1985) report successful land cover classification for the Sugar Creek watershed in Indiana using Landsat digital data. A grid cell data bank was developed for the watershed and satellite land cover information was used in assigning curve numbers for basin runoff modeling. Berich and Smith (1985) report similar use of Landsat data for modeling the 350 square mile Gunpowder Falls watershed in Maryland.

Without question, the technology that has changed flood forecasting most in the past decade is the microcomputer. In the last 10 years, computers have become smaller, more powerful, and affordable. With this new-found power has come a growing array of hydrologic modeling software designed for microcomputer users. Examples include the Large Basin Runoff Model which was developed by the Great Lakes Environmental Research Laboratory to forecast fluctuating water levels in the Great Lakes (Croley and Hartmann 1985).

The U.S. Army Corps of Engineers has also been actively involved in developing and adapting software for real-time data acquisition and analysis using small computers (Peters and Ely 1985, Clyde and O'Brien 1985). The rainfall-runoff component of this new software is HEC-1F, an adaptation of the widely used Corps software known as HEC-1.

To make HEC-1F applicable to real-time forecasting, several modifications have been made to make calibration and use faster and easier. Runoff is computed as the rainfall excess after an initial rainfall abstraction and a constant loss rate has been satisfied. Snyder unit hydrographs are used to obtain subbasin runoff hydrographs, and streamflow routing is done by the Muskingham Method. Automated parameter estimation is possible where corresponding rainfall and streamflow data records are available, and data entry has been simplified by allowing rainfall, loss rates, and baseflow parameters to be specified for aggregations of subbasins.

The Texas A&M Watershed Model is an interactive software package designed for use on an Apple IIe microcomputer (Rifai and Bedient 1985, Bell and James 1985). This model was developed for users with limited knowledge of hydraulics and hydrology. It features the SCS curve number approach for runoff estimation, a two parameter unit hydrograph, and variable storage coefficient streamflow routing. The standard step method is used to compute water surface profiles.

The major limitation of this model is that it relies on manually input radar data for basin precipitation estimates. Since radar provides only rough measurements of total precipitation, this limits the accuracy of the streamflow forecasts. Plans are underway to improve rainfall estimation by calibrating the radar data with limited real-time rain gage data.

New Modeling and Analysis Methods

Adaptive Hydrological Forecasting Rapidly growing demand for river forecasting in thousands of small flood hazard areas has focused attention on the need for improved basin modeling methods. The new models must be easily calibrated and applied, and they must make optimal use of large amounts of real-time hydrometeorological data now available through new instrumentation and communications technology.

One approach to these needs is through adaptive forecasting techniques based on concepts from time series analysis and control theory. According to Mehra (1980) and Wood (1980, p. 43), time series analysis was first introduced in the late 1920s. Since that time, associated analysis methods such as parameter optimization, multiple time series analysis via state-space methods, and Kalman filtering have been developed. Until recently, however, these tools have not been applied to hydrologic modeling.

No attempt will be made here to extensively review the literature on adaptive forecasting since this writer has had no exposure to the advanced statistical concepts that underlie these methods. A brief discussion of the general concept of adaptive forecasting and its benefits in flood forecasting is offered here simply to provide perspective on new techniques that are expected to play a growing role in future river forecasting operations.

Most of the major hydrologic models developed since the early 1970s have been deterministic in nature and conceptual in approach. Deterministic models, as defined by Fleming (1975, p. 316), are those which represent the processes of the hydrologic cycle quantitatively through mathematical functions.

Deterministic models may be either empirical or conceptual in approach. Empirical formulations utilize functions that lump many processes together, while conceptual models attempt to identify individual processes and their interrelationships. While empirical functions use relatively few parameters, their selection for a particular basin often requires considerable experience and personal judgement. Conceptual models frequently employ a larger number of parameters, but are more physically-based, making calibration less reliant on the personal judgement of the user.

A typical conceptual rainfall-runoff model, for example, simulates basin runoff processes such as interception, infiltration, overland flow, interflow, and channel flow as a network of storage compartments through which basin precipitation is routed. Generalized submodels for the various processes are tailored to match the characteristics of the particular basin being modeled through careful selection of the submodel parameters.

Although deterministic-conceptual hydrologic models have been quite effective for many river forecasting operations they have several deficiencies as noted by Chow, Watt, and Watts (1984). Because of the large number of parameters required by some models, they can be extremely time consuming and expensive to calibrate and use. These models are generally economically justified for river forecasting only where flood damage potential is high.

Another problem is that many of the more complex models were originally designed for long-term basin simulation rather than short-term forecasting. As a result, considerable time is needed to prepare input data and run the model. This can be a serious problem in small basins with short response times.

Finally, traditionally formulated conceptual models are not easily updated. As real-time data arrives at a forecast center it often is necessary to adjust or "update" the model if predicted flows do not agree with the observed streamflow. This is not unusual, particularly if a model was calibrated using a small number of historical events or if historical data used for calibration was inaccurate.

To update a traditionally formulated conceptual model to agree with real-time observations, it is necessary to arbitrarily adjust model input data or to reassess model parameters, neither of which is done easily or quickly.

To overcome these deficiencies, forecasters have begun to explore specially formulated adaptive forecasting methods based on concepts from time series analysis. The general concept of adaptive forecasting can be operationally defined by considering a basin for which real-time rainfall and streamflow measurements are available. In an adaptive forecasting mode, each new rainfall report is used to generate a new streamflow forecast. Each new forecast, in turn, is compared with real-time streamflow observations and the prediction error is fed back into the model to be used in reevaluating model parameters, adjusting input data, or doing both of these.

According to Chow, Watt, and Watts (1984), adaptive forecasting offers several important benefits:

1. Much of the information required to formulate a short-term forecast is contained in recent streamflow observations. Adaptive forecasting models make use of the information in real-time data by using it to update parameter estimates and model predictions;

2. The mathematical form of adaptive models tends to be relatively simple and computationally efficient, making them attractive for use on small computers; and

3. Forecasts are derived using recursive algorithms, thereby minimizing the computer storage needed for input and output variables.

Many different adaptive forecasting approaches have been developed. A common assumption of all methods, however, is that "noise" (a name for error) is inherent in measurements of model inputs and in real-time observations of the predicted variable. It is further assumed that noise has some stochastic structure which allows it to be predicted and fed back into a hydrologic model to improve the model or its output.

O'Connell and Clarke (1981) have presented a general review of adaptive hydrological forecasting techniques that apply least squares regression or composite transfer function-noise models to parameter optimization. Self-tuning algorithms which optimize the parameters of an error prediction function are also discussed.

Both Mehra (1980), and O'Connell and Clarke (1981) describe the general principles of state-space formulation of hydrologic models and use of Kalman filters to improve state estimation for hydrologic systems. State space formulations are among the most widely used adaptive forecasting techniques because of their flexibility. Both deterministic and stochastic models, that are either linear or non-linear, can be formulated in a state-space format.

A number of adaptive flood forecasting models have been tested. Examples include work by Chow, Watt, and Watts (1983, 1984) which included development and subsequent use of a model based on Box-Jenkins time series analysis. This model is being used to predict flooding on the Saint John River in New Brunswick. According to the authors, this project is believed to be the first application in real time of adaptive hydrologic forecasting.

Jones and Koch (1985) report successful testing of a flood stage estimation model applied on the Toutle and Cowlitz Rivers in Washington State. This flood routing model is based on state-space formulation of a conceptual system of cascading linear reservoirs. Kalman filtering was used to reduce errors in state estimation.

The potential for adaptive forecasting of spring snowmelt floods in the Sturgeon River basin was demonstrated by Burn and McBean (1985). Again, a state-space formulation and Kalman filtering were used.

Hydrometeorological Forecasting In urban basins with very short lag times, flash flood forecasts based on real-time precipitation measurements may not provide sufficient warning time for implementation of flood damage mitigation measures. To extend the warning time it becomes necessary to use rainfall forecasts as input to a hydrologic model.

Georgakakos and Bras (1984a, 1984b) have developed and tested a physically-based precipitation model that is designed to be linked with a hydrologic model. Ground level measurements of temperature, dew point temperature, and pressure are model inputs. The water equivalent mass of condensed vapor in a cloud column is the state variable. State-space formulation of the model and use of Kalman filtering permits updating using real-time precipitation measurements.

The Georgakakos-Bras precipitation forecasting model has been coupled with a hydrologic basin model to arrive at a hydrometeorologic flash flood forecasting model (Georgakakos 1986a, 1986b). In addition to the precipitation model, a modified version of the Sacramento Soil Moisture Accounting Model is used to simulate runoff and the processes that affect it. Channel routing is modeled as a series of nonlinear cascading reservoirs.

All elements of this hydrometeorological model are formulated in state-space form, and Kalman filtering is used to update model states using real-time observations of precipitation and streamflow. Real-time testing of this model in the Bird Creek basin in Oklahoma has yielded promising results.

PLAN OF STUDY

To achieve the objectives set forth earlier in this report, a three-phase plan of action was developed.

Phase I focused on identification of key hydrometeorologic features that characterize flooding in the Squaw Creek basin. Questions to be answered included:

1. How much basin-wide precipitation is necessary to cause flooding?
2. How much time lag occurs between a flood-producing rainfall and arrival of the flood crest at Ames?
3. How do storm duration and direction of travel influence streamflow? and
4. How do the various subbasins contribute to development of the flood crest?

To answer these questions, a basin model was created using HEC-1, a generalized computer model developed by the U.S. Army Corps of Engineers (U.S. Army Corps of Engineers 1985). Precipitation and streamflow data for the two largest floods of recent record were used to calibrate and verify the model. This calibrated model was then used to study basin response to various spatial and temporal storm patterns to identify critical hydrometeorological conditions that cause flooding.

Phase II of the project involved development of a low-cost, easy-to-use procedure for use by the city of Ames in predicting the magnitude and time of arrival of flood crests. Since this streamlined flood forecasting procedure was to be used by non-technical personnel, ease of application and interpretation of the resulting forecasts were primary objectives. At the same time, however, the simplified procedure had to be sufficiently sophisticated to recognize and predict the effects of critical spatial and temporal combinations of rainfall, soil moisture, and other key hydrometeorological features identified in Phase I. To test its performance, flood forecasts developed using the streamlined flood prediction procedure were compared with those from the more complex and detailed Phase I basin model.

The final phase of the project entailed development of guidelines and supplemental information for implementing a flood prediction program in Ames. In addition to providing recommendations for data acquisition, personnel, and equipment, it was necessary to create stage-discharge tables at critical flood damage centers in Ames so that depth of flooding can be estimated at these locations. The water surface profile model HEC-II, developed by the U.S. Army Corps of Engineers, was used to generate the desired stage-discharge information.

STUDY OF BASIN RAINFALL-RUNOFF RESPONSE

Model Development

Scope Limitations

As shown in Figure 5, floods in the Squaw Creek basin occur most frequently during the spring and summer months. Nearly one-third of the major events recorded since 1918 have occurred during the month of June. Clearly rainfall, rather than snowmelt, is the predominant cause of flooding in this basin. This fact, along with the extreme complexities of predicting floods caused by snowmelt and ice jams, lead to the decision to focus solely on rainfall-induced flooding. No attempt was made to model or forecast flooding caused by snowmelt.

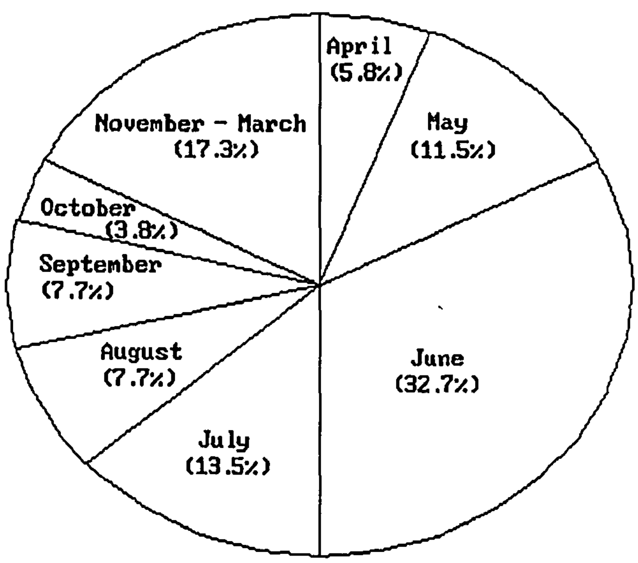

Figure 5. Month of major flood occurrence in Squaw Creek basin

General Model Selection

HEC-1, a generalized flood hydrograph software package developed by the U.S. Army Corps of Engineers, was selected for basin modeling. This choice was based on the following considerations:

1. The software was readily accessible through the facilities of the Iowa State University Computation Center;

2. The package offers deterministic, single-event modeling capabilities that are representative of current practices in basin modeling;

3. The Hydrologic Engineering Center of the Corps of Engineers supports the software and provides assistance and advice to users beyond that provided in written documentation (this proved valuable on two occasions); and

4. HEC-1 offers a variety of options for modeling major hydrologic processes and provides a reasonable compromise between empirical techniques and more complex and data-intensive models (Peters and Ely 1985, p. 7).

Modeling Concepts Using HEC-1

HEC-1, like other generalized river basin models, represents a basin as a conceptual network of subbasins, channels, and reservoirs. Hydrologic processes, including interception and infiltration, runoff,and channel flow, are represented by submodels consisting of general mathematical relationships which are tailored to simulate a particular subbasin or channel segment through selection of appropriate numerical parameters. In most instances, HEC-1 offers several optional submodels to represent each major hydrologic process.

The general structure and operation of an HEC-1 basin model can be visualized using the flow network shown in Figure 6. The basin to be modeled is broken into components--subbasins and channel segments--which are connected at nodes. Mathematical submodels that simulate subbasin processes, such as interception, infiltration, and runoff, transform precipitation on each subbasin into outflow hydrographs at each node. Individual hydrographs from subbasins and upstream channel segments are superimposed at each node to obtain a single composite hydrograph for the region upstream of the node. Routing models, which simulate the effects of channel storage and streamflow attenuation, transform a hydrograph at an upstream node into one at the adjoining downstream node.

The general procedure for developing the Squaw Creek basin model using HEC-1 was as follows:

1. Basin boundaries were mapped and the basin was divided into subbasins whose hydrometeorological characteristics were thought to be sufficiently uniform to be modeled by a single set of submodel parameters;

2. A conceptual basin model, consisting of a network of subbasins linked by stream channel segments, was developed (Figure 12);

3. Mathematical submodels were selected to simulate hydrologic processes in each subbasin and channel segment. Model parameters were selected to reflect hydrologic conditions in each subbasin and channel segment;

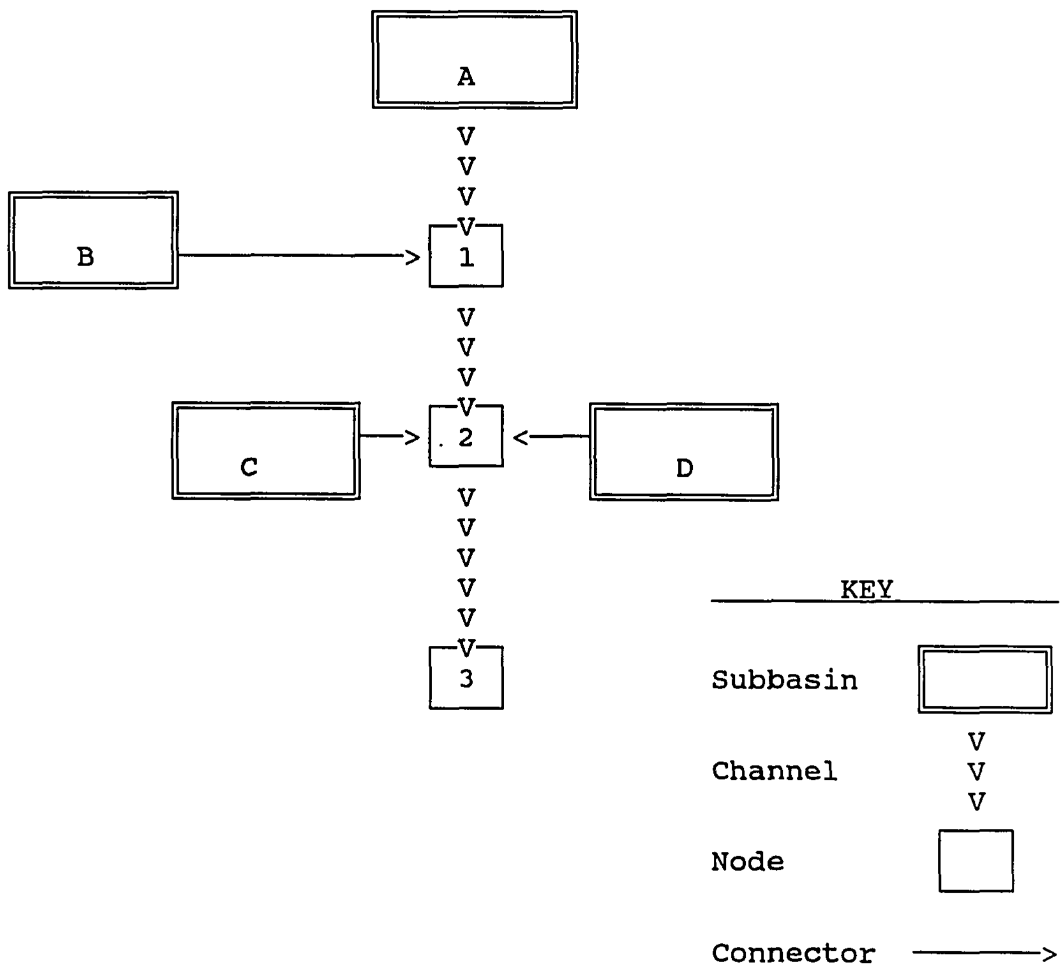

Figure 6. Conceptual representation of an HEC-1 basin model

4. The basin model was used to simulate the flood of June 1975, and predicted streamflow was compared with measured flow. Parameters were adjusted to bring predictions of peak flow, time of occurrence of the peak, and total runoff volume into line with measured values; and

5. The calibrated model was applied to three additional sets of precipitation data to verify that model predictions are in line with recorded streamflow for a variety of rainfall events.

A more detailed description of these procedures is given in following sections of this report.

Basin and Subbasin Delineation

The first task in modeling the Squaw Creek basin was to identify major components--subbasins and connecting channels in this case--which characterize the basin. Although it is conceivable that an area the size of the Squaw Creek basin might be modeled as a single basin for some purposes, this was not appropriate for this study for two reasons:

1. Observations during the 1975 flood suggested that basin response might be related to direction of storm travel and timing of precipitation at various locations throughout the basin. If the basin is not broken into sub-parts, it would be impossible to model the effects of spatial and temporal variations in rainfall; and

2. Topographic maps of the basin showed that some regions were extremely flat, with little natural drainage, while others were rolling and drained by well-defined streams. These two types of terrain were expected to respond differently to heavy rainfall.

Based on the considerations listed above, it was decided to break the basin into subbasins based on manmade and natural drainage patterns. A review of U.S. Geological Survey (USGS) topographic maps (7 1/2 minute quadrangles 1:24000 scale) showed upland areas in the northern and eastern portions of the basin to be extremely flat (Figures 7 and 8). The headwaters of the basin are derived largely from agricultural drainage district mains and drainage ditches (Figures 9 - 11). Further to the south natural tributaries are found, but upland areas in these subbasins are also quite flat and heavily reliant on subsurface drain tile systems to make them tillable.

Because of the broad flat uplands, it was impossible to locate basin and subbasin boundaries using the ten-foot interval contour lines provided on USGS topographic maps. To aid delineation of these boundaries, drainage district maps were solicited from county engineers in Webster, Hamilton, Boone, and Story Counties. Although these maps did not contain topographic information, they did locate tile mains sufficiently to infer flow patterns and drainage divides.

Using drainage district maps in conjunction with USGS topographic maps, it was possible to divide the Squaw basin into 13 subbasins as shown in Figure 12. These were outlined on topographic maps and planimetered to determine their areas. Their combined drainage area was determined to be 226.93 square miles; a figure which compared favorably with the 227 square mile area listed for the Squaw Creek basin in

Figure 7. Extremely flat terrain characterizes the northern Squaw basin in Hamilton County (Section 34 Webster Township)

Figure 8. View looking SW from the eastern boundary of the Squaw basin. The grain elevator on the horizon is at the western edge of the basin 11 miles away.

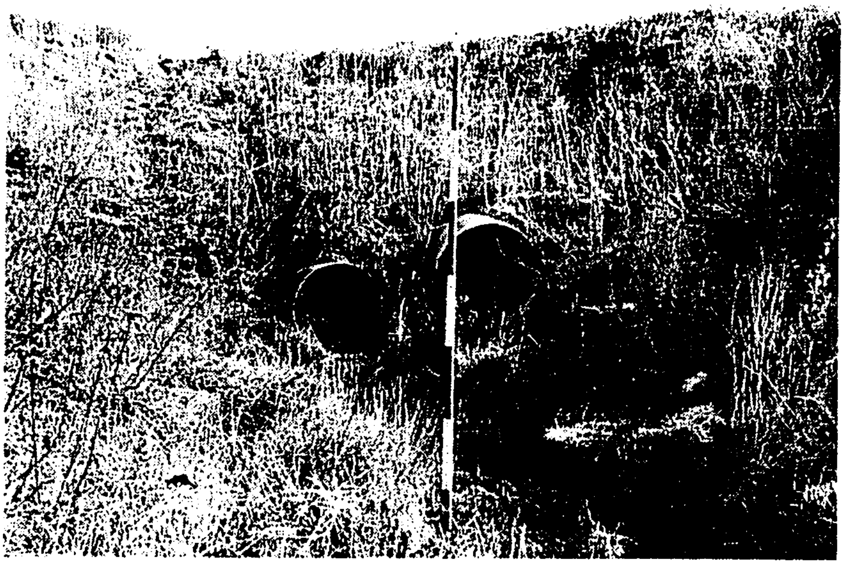

Figure 9. Drainage mains in Hamilton County are the source for Squaw Creek (S33,T-87N,R-26W)

Figure 10. This deep drainage ditch forms the main Squaw Creek channel west of Stratford on Iowa Highway 175

Figure 11. Subsurface tile drainage lines contribute to streamflow throughout the basin

Drainage Areas of Iowa Streams (U.S. Geological Survey 1957, p. 373). Table 4 lists the subbasins and their respective areas.

Once the basin was divided into subbasins the overall structure and operation of the model was established. In this case it consists of 13 subbasins linked by six stream channel segments as shown by the model schematic in Figure 13. It should be noted that the stream gaging station on Squaw Creek lies above the confluence with Worrel Creek (subbasin G). For this reason, subbasin G was not included during Phase I model development since historical streamflow data used for model calibration and verification does not reflect the Worrel Creek flow contribution. The Worrel Creek

Table 4. Squaw Creek subbasins and their areas

Subbasin Label [a]	Associated Stream Name	Area (sq. miles)
A	Squaw Creek	18.29
B1	Unnamed tributary	16.78
B2	Crooked Creek	22.77
C1	Unnamed tributary	16.70
C2	Unnamed tributary	14.42
D1	Montgomery Creek	24.66
D2	Prairie Creek	14.77
D3	Lundy's Creek	9.46
D4	Unnamed tributary	17.01
E1	Onion Creek	26.70
E2	Unnamed tributary	16.56
F	Clear & College Creeks	14.27
G	Worrel Creek	14.54
	Total	226.93

[a]Subbasins are labeled in alphabetical order from north to south as shown in Figure 12.

subbasin was added to the model after calibration and verification by applying the same submodel parameter selection procedures used for subbasins further upstream.

Submodel Selection and Parameter Estimation

Each subbasin in the conceptual model illustrated in Figure 13 requires two submodels: one to estimate the quantity of runoff caused by precipitation during each computational interval, and one to transform the runoff into a subbasin outflow hydrograph.

Similarly, each channel segment requires a submodel to transform subbasin outflow hydrographs (and hydrographs from

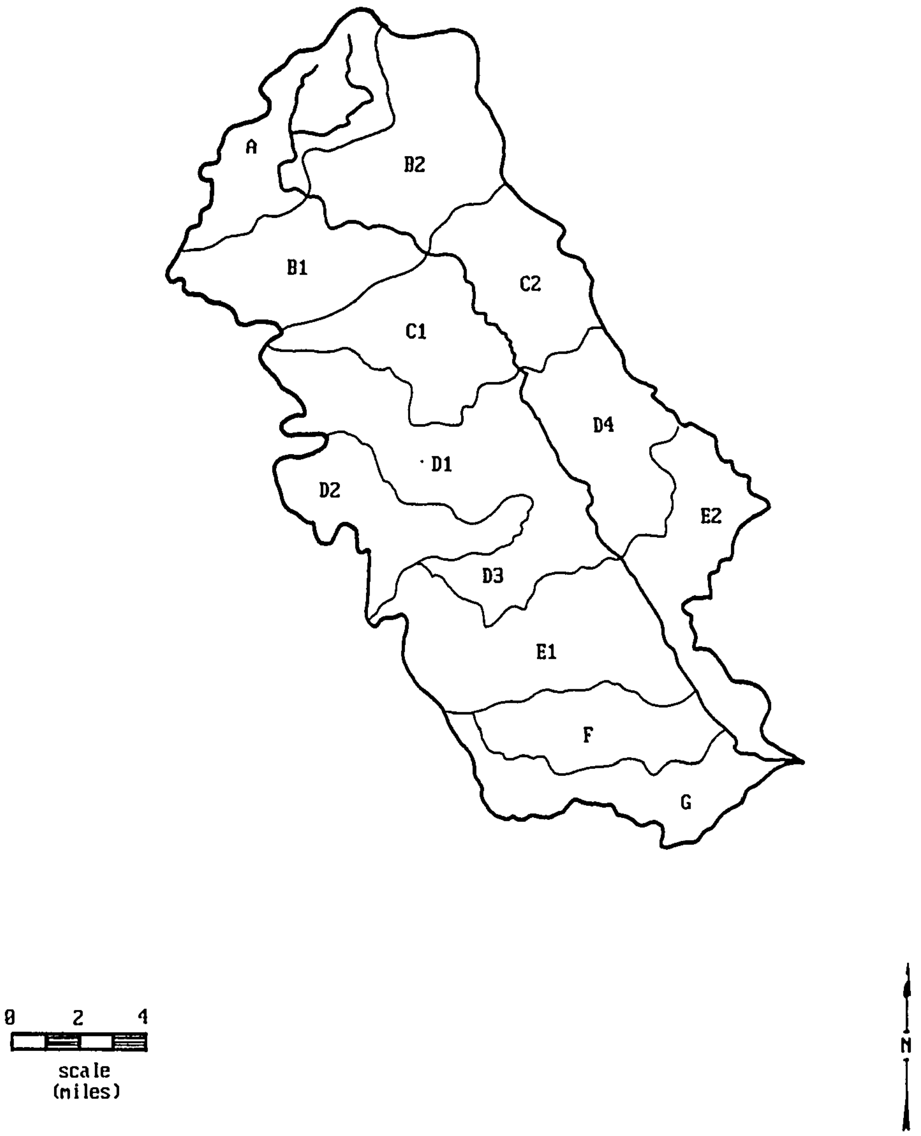

Figure 12. Squaw basin map showing subbasins

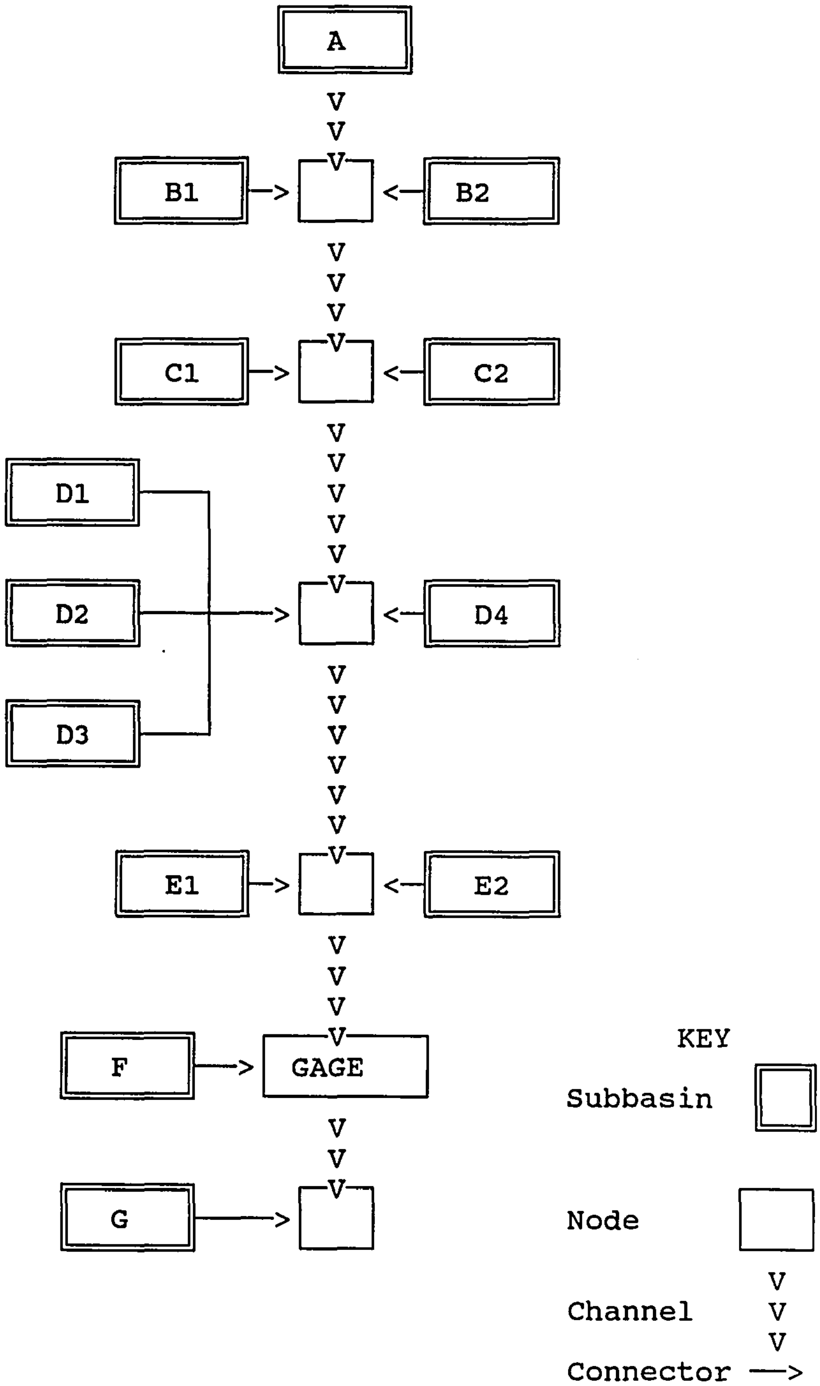

Figure 13. Schematic of Squaw Creek basin model

the adjoining upstream channel segment) at upstream nodes into an outflow hydrograph at the downstream node. The following sections discuss the rationale for submodel selection and the procedures for obtaining initial parameter estimates used in model calibration.

Runoff Modeling The amount of surface runoff generated by a specific quantity of rainfall depends on how much precipitation is lost from the runoff process to interception, depression storage, and soil infiltration. HEC-1 calculates these losses for each computational time interval during a storm, and subtracts them from precipitation occurring during that interval to derive a runoff estimate.

The rate of precipitation loss declines throughout the storm as interception, depression, and soil moisture storage reservoirs become filled. HEC-1 offers four submodels for simulating rainfall losses during the course of a storm: a constant loss rate model, an exponentially declining model, the Soil Conservation Service (SCS) curve number model, and the Holtan model.

The uniform loss rate model was not considered since it cannot accurately simulate the declining precipitation loss rate during an extended storm.

The exponential and Holtan loss rate models both require selection of four separate empirical parameters.

Considerable practical experience in using these models, or a substantial program of field data collection, would be necessary to determine reasonable parameter values for the 13 Squaw Creek subbasins. Since experience using these models was lacking, and there was neither time nor money for field data collection, use of the exponential and Holtan models was rejected.

The SCS curve number method for estimating runoff is an empirical procedure designed for use on ungaged watersheds. Runoff estimates are easily obtained using rainfall, soil type, and land use data that are readily available for most watersheds. For these reasons, the SCS method was selected for use in the Squaw Creek basin model.

Runoff estimation is based on the following equations (U.S. Dept. of Agriculture 1972, pp. 10.4-10.5):

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \qquad (4)$$

where:

Q = Basin runoff in inches;

P = Total storm rainfall in inches;

S = Potential maximum abstraction in inches;

I_a = Initial abstraction in inches (the amount of rainfall that must occur before any runoff is generated).

Furthermore, S is related to the curve number (CN) by:

$$S = \frac{1000}{CN} - 10 \qquad (5)$$

and the initial abstraction (I_a) is estimated from the following empirically observed relationship:

$$I_a = 0.2\ S \qquad (6)$$

Since both S and I_a can be estimated from the curve number, CN is the only parameter needed to calculate Q using the SCS method. Curve numbers range from 100, for very smooth impermeable surfaces that retain no precipitation, to zero for surfaces that, theoretically, can retain all precipitation.

Curve number selection is based on soil type, land use (cropland, pasture, forest, residential, commercial, etc.), and soil moisture. To aid CN selection, the SCS has classified every soil type into one of four hydrologic soil groups--labeled A,B,C, or D--based on results of runoff studies conducted on small watersheds. Soils in group A have the greatest potential to retain precipitation and have low curve numbers. Soils in group D retain precipitation the least, and have higher curve numbers. Soils with intermediate runoff characteristics are assigned to hydrologic soil groups B or C.

Land use considerations are factored into CN selection using Table 9.1 of Section 4 of the National Engineering Handbook (U.S. Dept of Agriculture 1972, p. 9.2) which lists curve numbers for various combinations of land use and hydrologic soil group. Land in hydrologic soil group A, for example, is assigned a CN of 72 (assuming moderate soil moisture) when used for row crop production. The same land used as pasture, however, will exhibit less runoff and is assigned a CN of 39.

Since dry soils retain more precipitation than wet ones, the effects of soil moisture must also be accounted for when assigning curve numbers. To avoid the need for field soil moisture measurements, the SCS method defines three general soil moisture categories based on the amount of rainfall accumulated during the five-day period prior to the storm for which a runoff estimate is desired. During the growing season, rainfall accumulations less than 1.4 inches are classed as Antecedent Moisture Condition I (AMC I). AMC II includes five-day rainfall amounts ranging from 1.4 to 2.1 inches. Rainfall totals exceeding 2.1 inches in five days are classed as AMC III (U.S. Dept. of Agriculture 1972, p. 4.12).

To simulate variability in runoff caused by changing soil moisture conditions, the SCS method adjusts the curve number. A group B soil in row crop cultivation is assigned a CN of 81 under moderate soil moisture conditions (AMC II). When the same soil is dry (AMC I) a curve number of 64 is assigned, and under wet conditions (AMC III) the curve number is 92.

Table 5. Major soil associations in the Squaw Creek basin and their curve numbers

Soil Association	Weighted Curve Number
Clarion-Webster-Nicollet	81[a]
Clarion-Storden-Coland	71
Hayden-Lester-Storden	60
Coland-Spillville-Zook	81
Canisteo-Okoboji-Nicollet	80
Clarion-Zenor	77
Canisteo-Clarion-Nicollet	80
Brownton-Ottosen-Bode	81
Canisteo-Clarion-Nicollet	80
Marna-Guckeen	87
Webster-Clarion-Nicollet	80

[a]Curve numbers are for AMC II.

To select appropriate curve numbers for the Squaw Creek subbasins, it was necessary to identify the predominant soil types. Soil maps for the basin (U.S. Dept. of Agriculture 1975, 1981, 1984, 1986) showed hundreds of mapping units in each subbasin. Clearly, mapping and planimetering these small areas was not practical. As an alternative, major soil associations were sketched on the basin map and an areally weighted average curve number was assigned to each major soil association. Curve number weighting was based on SCS estimates of the percentage of the major soil association areas typically occupied by various soils. Table 5 lists major soil associations in the Squaw Creek basin and the weighted curve number assigned to each.

Once a curve number was assigned to each major soil association, the area of each soil association in a subbasin was planimetered and an areally weighted curve number for the subbasin was calculated. Table 6 indicates the average curve numbers assigned to each subbasin.

Table 6. Squaw Creek subbasins and their assigned SCS curve numbers

Subbasin	AMC I	Curve Number AMC II	AMC III
A	64	81	92
B1	64	81	92
B2	64	81	92
C1	60	78	90
C2	63	80	91
D1	62	79	91
D2	63	80	91
D3	63	80	91
D4	62	79	91
E1	60	78	90
E2	62	79	81
F	59	77	89
G	59	77	89

Hydrograph Development   After a runoff volume estimate for each computational interval is obtained, a second subbasin model is required to transform that runoff volume into an outflow hydrograph.

Since no rainfall or streamflow data were available for any of the 13 Squaw Creek subbasins, it was impossible to develop subbasin unit hydrographs based on historical data. For this reason, it was necessary to use synthetic unit hydrographs whose height and shape are related empirically to subbasin parameters such as area and time of concentration.

HEC-1 offers three synthetic unit hydrograph options: the Clark Method, the Snyder Method, and the SCS Dimensionless Method.

The Clark Method derives a crude subbasin outflow hydrograph from user-supplied time-area data that indicates the fraction of the subbasin contributing to outflow at various times during a runoff event. This outflow hydrograph is refined by routing it through a linear reservoir to simulate basin storage effects on runoff. To apply the Clark Method the user must supply subbasin time of concentration, time-area data, and a subbasin storage factor used to derive routing coefficients. If time-area data are not available, HEC-1 supplies an empirical relationship to generate these data. Since no outflow hydrographs were available for any of the Squaw Creek subbasins, estimated time-area data and storage coefficients would be very crude at best. For this reason, the Clark Method was not considered for use in the Squaw Basin Model.

The Snyder Method was originally developed from runoff studies in the Appalachian highlands. This method does not produce a complete unit hydrograph. Instead, it determines peak discharge, time to peak, and the width of the unit graph at 50 and 75 percent of the peak discharge. To obtain a complete hydrograph using the Snyder Method, HEC-1 combines elements of the Snyder and Clark methods to obtain a complete unit hydrograph with Snyder characteristics. Because of the questionable relevance of the Snyder Method to central Iowa conditions, and the previously discussed concerns about the Clark Method, the Snyder Method seemed to offer little to benefit the Squaw Basin Model.

The SCS Dimensionless Unit Hydrograph Method is the most easily used of the three synthetic methods offered by HEC-1. The only input parameters required are basin area and time of concentration. From these, peak discharge, time to peak, and the complete shape of the unit hydrograph can be derived using the SCS Method.

As will be shown below, the simplicity of the SCS Method stems from its reliance on empirical relationships. These were developed from watershed studies throughout the country, and the SCS method is widely used for small structure design throughout the Midwest. For this reason, the SCS Method was felt to be the best choice for development of the Squaw Basin Model.

The SCS Dimensionless Unit Hydrograph Method is formulated (U.S. Dept. of Agriculture 1972, pp. 10.6-10.7) as follows:

$$q_{pk} = \frac{484\ A}{T_{pk}} \tag{7}$$

$$T_{pk} = \frac{Delt}{2} + T_{lag} \tag{8}$$

$$T_{lag} = 0.6 * T_c \tag{9}$$

where:

q_{pk} = Unit hydrograph peak discharge in cubic feet per second;

A = Basin area in square miles;

T_{pk} = Time to peak in hours;

Delt = Duration of unit hydrograph in hours (SCS recommends Delt <= 0.25 * T_{pk});

T_{lag} = Basin lag (in hours) defined as the time between the center of mass of the excess rainfall hyetograph and the time of occurrence of the unit hydrograph peak;

T_c = Basin time of concentration in hours.

To apply the SCS Method, one begins with an estimate of basin time of concentration (T_c) and calculates T_{lag} from equation 9. Using equation 8, T_{pk} can be calculated, and q_{pk} is estimated using equation 7.

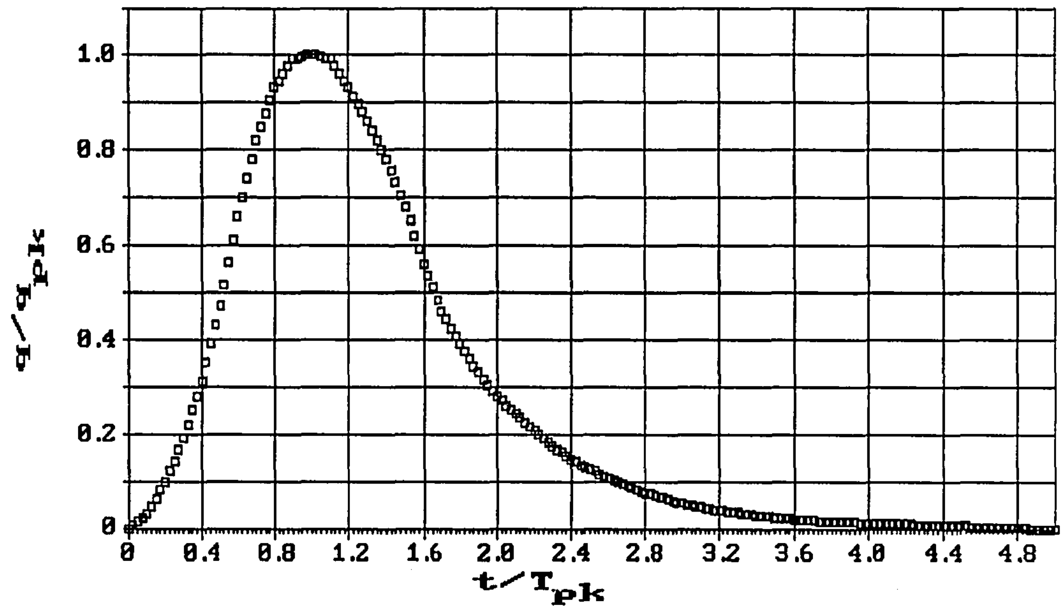

Figure 14. SCS dimensionless unit hydrograph

The complete unit hydrograph, drawn using the SCS dimensionless unit graph of q/q_{pk} versus t/T_{pk}, is shown in Figure 14. To determine the unit graph discharge q at any time t after initiation of runoff producing rainfall, the fraction t/T_{pk} is calculated and the corresponding ratio of q/q_{pk} is read from the SCS graph.

For large basins, the SCS recommends that time of concentration be calculated from estimates of flow distance and average channel and overland flow velocity (U.S. Dept. of Agriculture 1972, pp. 15.1-15.16). Channel flow distances are scaled from subbasin maps. Overland flow distances are measured along the longest possible flow path which can be sketched from the upper end of the channel to a point on the subbasin boundary.

The SCS suggests that channel velocities be calculated using bankfull discharge for the low-flow channel, or by using a discharge with a two-year return period. Estimated channel geometry and the Manning Equation are used to calculate flow velocity.

For the purpose of this study, rough estimates of channel geometry at four locations in the basin were obtained from channel cross sections taken on bridge design sheets provided by the Iowa Department of Transportation. These data were supplemented by black and white photographs taken from Iowa Highway 17, which crosses many Squaw Creek tributaries in the western half of the basin. A range pole driven into the stream bottom provided a rough photographic scale for estimating channel width (Figures 15 - 18). Since only low flow channel geometry was necessary to make the velocity estimates, a trapezoidal channel with one-on-one side slopes and bottom widths of 10 to 20 feet adequately described the tributary streams in most cases.

An estimate of the two-year frequency discharge for the major stream in each subbasin was obtained using regression equations from <u>Floods in Iowa: Technical Manual for Estimating their Magnitude and Frequency</u> (U.S. Dept. of the

Figure 15. Main channel of Squaw Creek at Iowa Highway 17

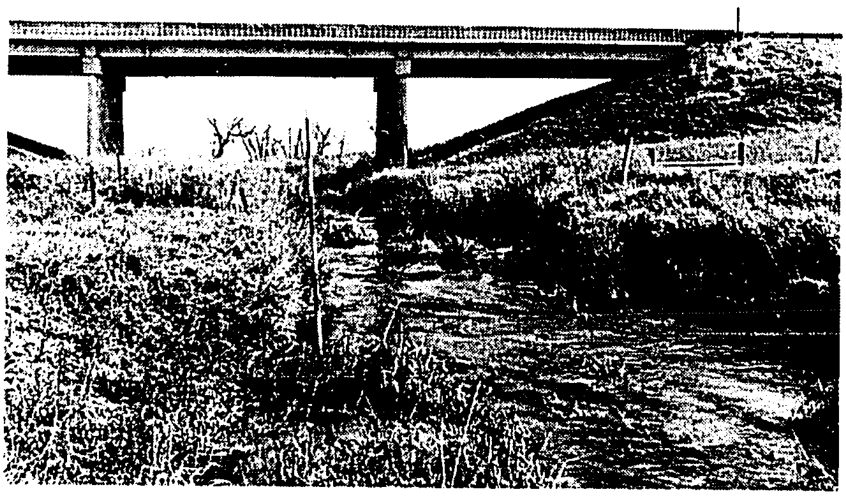

Figure 16. Montgomery Creek at Iowa Highway 17

Figure 17. Prairie Creek at Iowa Highway 17.

Figure 18. Onion Creek at Boone/Story County line

Interior 1973). For northcentral Iowa, the two-year frequency discharge is estimated from:

$$q_2 = 41.9\ A^{0.672} \tag{10}$$

where:

q_2 = Two-year frequency discharge in cubic feet per second;

A = Basin area (square miles) upstream of the discharge point.

Manning's Equation, formulated for a trapezoidal channel, was solved iteratively to determine depth and cross sectional area of flow. The continuity equation was applied to determine average flow velocity for the two-year discharge rate. A Manning's roughness value of 0.03 was assumed for channel flow. Average channel slope in each subbasin was determined using the following equation (U.S. Dept. of the Interior 1973, p. 41):

$$S_{av} = \frac{E_{85} - E_{10}}{0.75\ L} \tag{11}$$

where:

S_{av} = Basin average slope in feet per foot;

L = Length of channel in feet;

E_{85} = Elevation of channel (feet) at a location 0.85 L upstream from the channel mouth;

E_{10} = Elevation of channel (feet) at a location 0.10 L upstream from the channel mouth.

Overland flow velocities were estimated using a graph of velocity versus land slope (for various land cover classifications) which was developed by the SCS (U.S. Dept. of Agriculture 1972, pp. 15.8). Overland flow distance was measured along the longest flow path extending from the upper end of the stream channel to a point on the basin divide. Average overland flow slope was calculated using equation 11. Row crops were assumed to be the predominant land use.

Table 7 summarizes travel time calculations for overland and channel flow in each subbasin, and the resulting times of concentration.

Table 7. Summary of time of concentration calculations for Squaw Creek subbasins

	Channel Flow			Overland Flow			
Basin	Length (ft)	V (ft/s)	Travel Time (hrs)	Path Length (ft)	V (ft/s)	Travel Time (hrs)	Time of Conc. (hrs)
A	29800	2.8	3.0	10000	0.40	6.9	9.9
B1	11200	4.3	0.7	18000	0.40	12.5	13.2
B2	33500	4.0	2.3	10600	0.50	5.9	8.2
C1	38600	4.5	2.4	5200	0.60	2.4	4.8
C2	18200	4.5	1.1	8800	0.60	4.1	5.2
D1	56000	4.0	3.9	17200	0.40	11.9	15.8
D2	59000	3.8	4.3	3200	0.50	1.8	6.1
D3	19200	5.3	1.0	17000	0.65	7.3	8.3
D4	22400	5.3	1.2	7400	0.75	2.7	3.9
E1	63600	4.6	3.8	10800	0.75	4.0	7.8
E2	18000	4.8	1.0	22200	0.50	12.3	13.3
F	33200	4.5	2.0	20000	0.65	8.5	10.5
G	48000	5.4	2.5	14800	0.60	6.8	9.3

It is worth noting that initial estimates of subbasin times of concentration--and the resulting lag times computed from them--were considerably lower than those shown in Table 7. As a result, substantial difficulty was encountered during early attempts to calibrate the Squaw Basin Model. Projected flood peaks tended to arrive early and have a higher crest than indicated by historic streamflow records. Consultation with Drs. T.A. Austin and Ronald Rossmiller, from the Department of Civil Engineering at Iowa State University, suggested that the estimated subbasin times of concentration appeared to be low, although the estimating procedures seemed reasonable.

A Saturday afternoon auto tour of several subbasins revealed the primary cause of this difficulty. Field observations showed that U.S. Geological Survey topographic maps tended to show well-defined stream channels in upland areas of the subbasins where flow paths are actually barely discernible. This meant that estimated channel flow distances were overstated by a mile or more in some instances, and that overland flow distances were underestimated. The photograph in Figure 19 shows an area mapped as a stream channel on a USGS topographic map. Note that not even a grassed waterway is evident in this field.

After discovering this systematic error in the time of concentration estimates, an alternate procedure for

Figure 19. Upland region of subbasin E2 (SW 1/4 S 33, T-85N, R-24W) showing watercourse mapped as a "stream"

estimating channel and overland flow distances was sought. Review of SCS Soil Survey maps for the Squaw basin showed that these maps differentiate between waterway segments that are crossable with tillage equipment and those that are not. Furthermore, the point of transition from uncrossable to crossable seemed to agree reasonably well with field observations of where well-defined channels ended in upland areas. These transition points were located on the subbasin maps, and channel and overland flow distances were redefined from these locations. This shortened channel flow distances and increased overland flow path lengths, resulting in an overall increase in estimated time of concentration. This substantially improved the timing and magnitude of flood crest projections for the calibration event.

To check the time of concentration estimates shown in Table 7, several empirical relationships were investigated. Hall and Austin (1980) have reviewed 18 different methods for estimating basin time parameters using physical characteristics such as slope, drainage area, and travel distance. As shown in Table 8, however, these empirical methods have practical limitations imposed by special characteristics of the watershed studies they were derived from, or by the way in which they are formulated.

Limits based on geographic location and drainage area are common. In some instances the relationships rely on selection of basin-specific coefficients, making their use difficult unless previous experience provides a basis for selecting coefficients that are representative of local conditions. Based on the practical limitations shown in Table 8, the Mitchell Method, which was developed from studies of relatively large watersheds in Illinois, seemed to be the only empirical method well suited for use in the Squaw Creek basin.

Mitchell's work was based on regression analysis of hydrographs from 58 gaged watersheds in Illinois (Illinois

Table 8. Comparison of empirical equations for time of concentration and lag (Hall and Austin, 1980)

Method	Application Limits
Time of Concentration Equations	
Kirpich	Drainage areas < 200 acres.
Pickering	Drainage areas < 200 acres.
Mockus	Drainage areas < 200 acres.
SCS	Drainage areas < 800 acres.
Singh	Derived from kinematic wave theory--has strong theoretical base but little field verification--requires rainfall intensity data.
Kerby	Assumes overland flow distances < 1200 feet--applicable only to overland flow--requires selection of retardance coefficient that ranges from 0.02 - 0.80.
Kinematic wave	Valid only for overland flow on homogeneous surfaces--requires rainfall intensity-duration-frequency data.
Federal Aviation	Developed for use on airport runways.
Lag Time Equations	
Snyder	Developed from runoff studies in Appalachian highlands--must select watershed characteristic coefficient ranging from 0.7-2.2 .
SCS Lag	Drainage areas < 2000 acres.
Hickock-Keppel-Rafferty	Developed from data on semi-arid range land with drainage areas < 800 acres.

Table 8. (continued)

Method	Application Limits
Mitchell	Developed from runoff data on 58 Illinois watersheds with drainage areas ranging from 10 - 3090 square miles.
Bureau of Reclamation	Requires selection of two empirical coefficients.
Taylor-Schwarz	Developed from 20 watersheds in North and Middle Atlantic states with drainage areas from 20 - 1600 square miles--requires large amount of data on stream geometry and slope.
SCS Incremental	Treats lag as a weighted time of concentration from elemental areas of the basin--must divide basin into small elements and calculate travel time to basin outlet for each.
Eagleson	Applicable only to sewered watersheds with drainage areas < 8 square miles.

Dept. of Public Works and Buildings 1948). Care must be taken when comparing lag time estimates derived using Mitchell's work with estimates made using SCS methods since Mitchell's definition of lag differs from that of the Soil Conservation Service. By Mitchell's definition, lag is the time, in hours, from the center of mass of the excess rainfall hyetograph to the center of mass of the runoff hydrograph. The SCS, however, measures lag from the center of mass of excess rainfall to the time when peak flow occurs.

Using regression analysis Mitchell derived the following equation for lag time:

$$T_{lag} = 1.05\ A^{0.60} \tag{12}$$

where:

T_{lag} = Lag time in hours (Mitchell's definition);

A = Drainage area in square miles.

Table 9 compares subbasin lag estimates derived using Mitchell's equations with those obtained by applying equation 9 to the time of concentration estimates in Table 7. Although they do not agree in every case, the two estimates compare reasonably well. Lag times based on Mitchell's equation are the larger of the two estimates in slightly over half of the basins. This is not surprising since Mitchell's

Table 9. Estimates of subbasin lag time

	Lag Estimate (hours)		
Subbasin	SCS Method	Mitchell's Method	
A	5.9	6.0	3.8 - 8.2[a]
B1	7.9	5.7	3.6 - 7.8
B2	4.9	6.8	4.3 - 9.3
C1	2.9	5.7	3.6 - 7.8
C2	3.1	5.2	3.3 - 7.1
D1	9.5	7.2	4.5 - 9.9
D2	3.7	5.3	3.3 - 7.3
D3	5.0	4.0	2.5 - 5.5
D4	2.3	5.7	3.6 - 7.8
E1	4.7	7.5	4.7 -10.3
E2	8.0	5.1	3.2 - 7.0
F	6.3	5.2	3.3 - 7.1
G	5.6	5.2	3.3 - 7.1

[a]Range based on Mitchell's estimate of probable error of 37.4 % in time of crest.

definition of lag is measured with respect to the center of mass of a hydrograph instead of its peak. Since hydrographs tend to be skewed to the right, their centers of mass would be expected to lie to the right of the peak.

Baseflow Modeling HEC-1 does not offer a separate deterministic submodel for calculating groundwater contribution to streamflow. Instead, the effects of baseflow are simulated by artificially altering the shape of the falling limb of the runoff hydrograph to match that exhibited by streamflow records from past events.

Two parameters--the baseflow recession coefficient(K), and the baseflow initiation point (q_0)--control the simulated magnitude and rate of decline of subbasin baseflow. HEC-1 computes baseflow using the following exponential depletion equation:

$$q_t = \frac{q_0}{K^{(n*delt)}} \qquad (13)$$

where:

q_t = baseflow discharge (cubic feet/second) at time n*delt hours since baseflow was initiated;

q_0 = Discharge (cubic feet/second) at which baseflow is initiated;

K = Baseflow recession coefficient;

delt = Length (in hours) of computational interval;

n = Number of computational intervals since baseflow was initiated.

HEC-1 allows q_0 to be specified by indicating the fraction of the peak flow at which baseflow is initiated. The effects of changing q_0 are demonstrated in Figure 20 where baseflow is initiated at both 10 percent and 20 percent of peak flow. Figure 21 illustrates the impact of changing the recession coefficient (K) from 1.01 to 1.04 while holding q_0 constant.

While the effects of the two baseflow parameters on the shape of a single subbasin hydrograph are easily predicted, their impacts on a composite hydrograph, formed by summation of two hydrographs, are more difficult to anticipate.

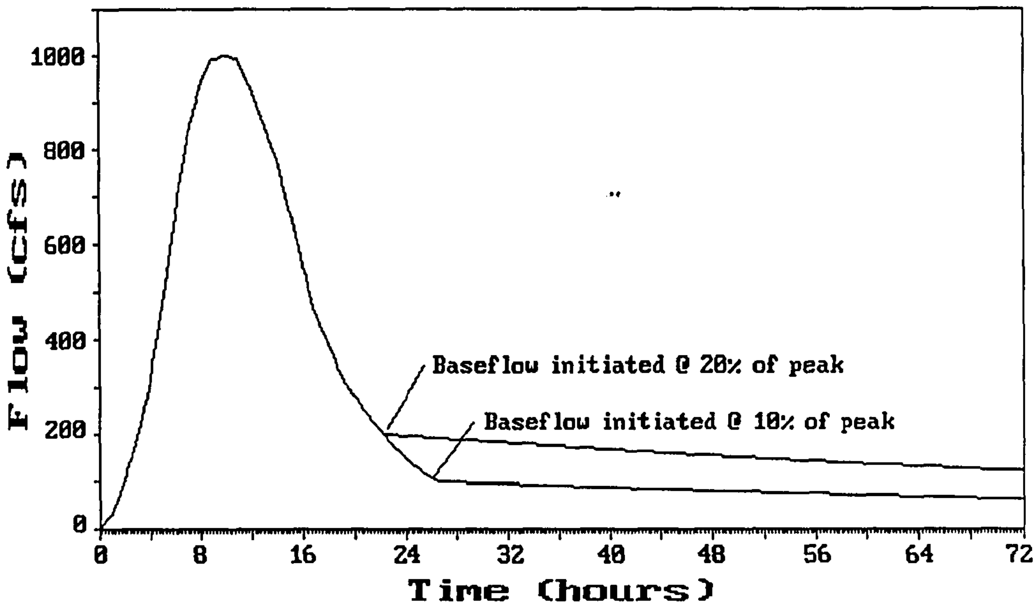

Figure 20. Effects of changes in q_0 on runoff hydrograph

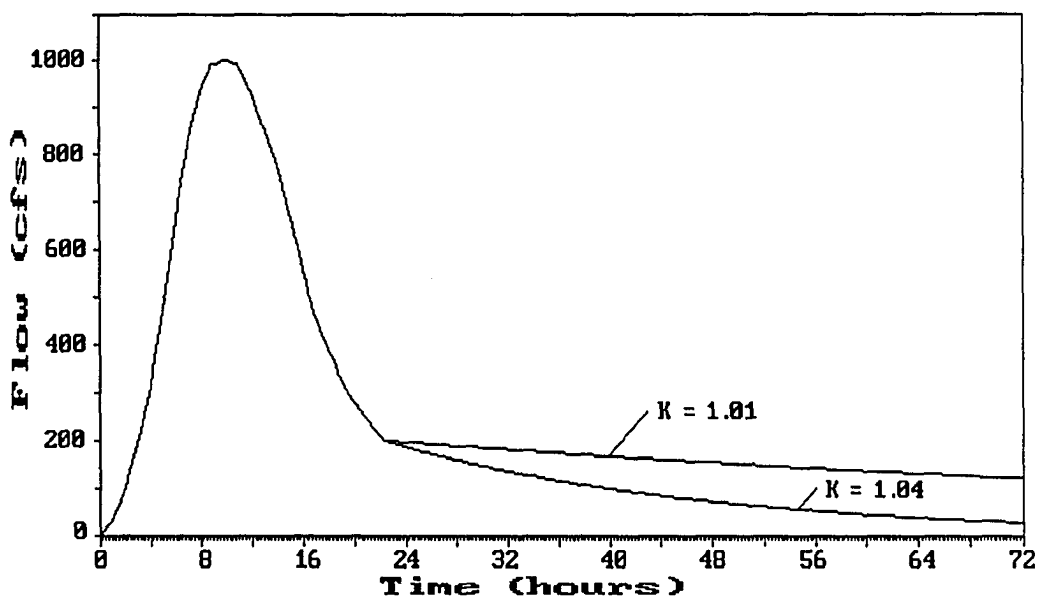

Figure 21. Effects of changing K on runoff hydrograph

Figure 22 illustrates the summation of two hydrographs whose peaks are separated by several hours--as in the case of two subbasins contributing to a stream at different locations. Note that, while both subbasin hydrographs initiate baseflow at 20 percent of their respective peak flows, baseflow recession for their sum occurs at nearly 40 percent of the peak flow.

Clearly, the amplitude, shape, and timing of superimposed hydrographs can significantly affect the baseflow recession characteristics of their summation. For this reason subbasin recession coefficients, particularly q_0, cannot be selected solely on the basis of recession characteristics of the total basin.

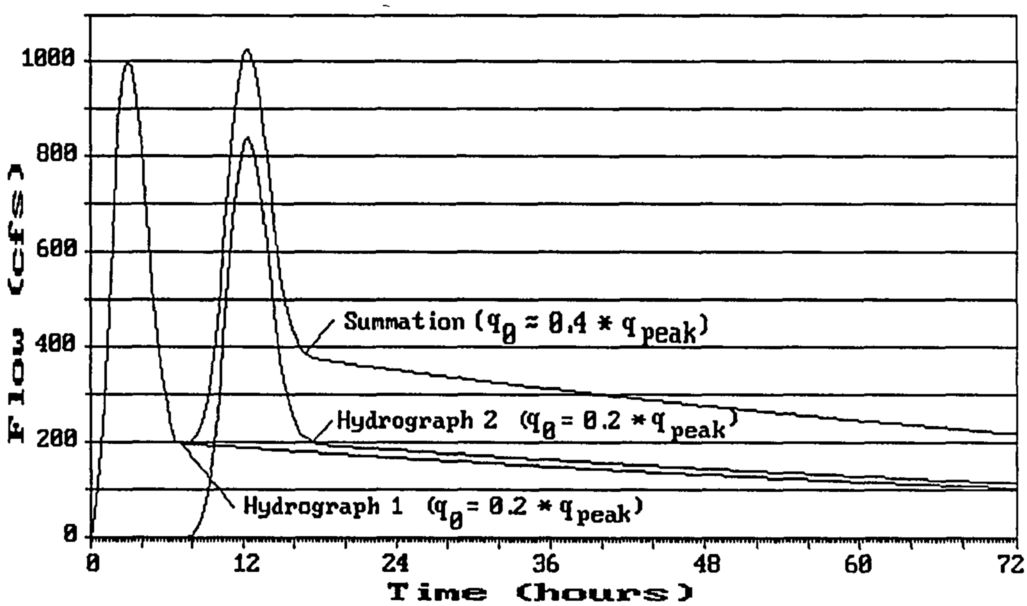

Figure 22. Effects of hydrograph summation on apparent baseflow recession

Since the only streamflow data available were for the total basin, however, daily average streamflow during periods of little or no recorded rainfall in 1975 and 1983 were analyzed to obtain rough estimates of q_0 and K for the total basin. These estimates were used as a starting point for assigning subbasin recession constants during model calibration.

As shown in Figures 23 and 24, log q_t was plotted against time (in days) since the most recent flow peak. A straight line was sketched through the portion of each flow sequence having the lowest slope. Taking the log of both sides of equation 13 gives:

$$\log q_t = \log q_0 - \log K(n*delt) \qquad (14)$$

which shows -log K to be the slope of a linear relationship between log q_t and n*delt (note that slope must be computed using time in hours, rather than days, since K--as defined in equation 13--is an hourly decay coefficient).

The K values for seven recession sequences shown in Figures 23 and 24 ranged from 1.002 to 1.006 with values around 1.004 being most common. In his study of low flow recession patterns at 76 stream gaging stations throughout Iowa, Howe (1968) reported recession constants of 1.003 to 1.006 for basins with areas less than 100 square miles. For

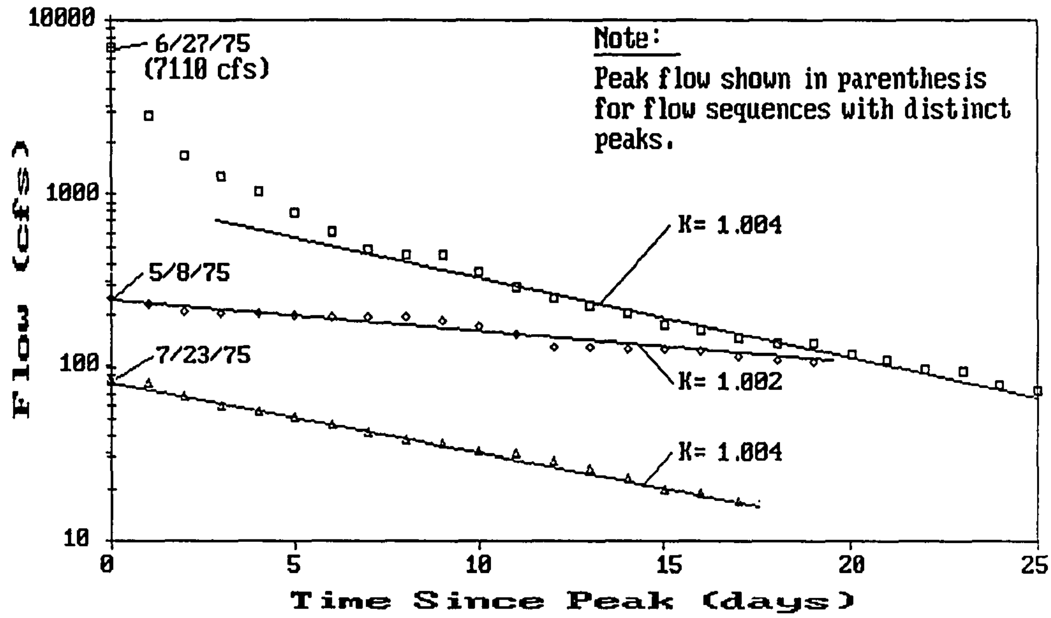

Figure 23. Baseflow recession on Squaw Creek in 1975.

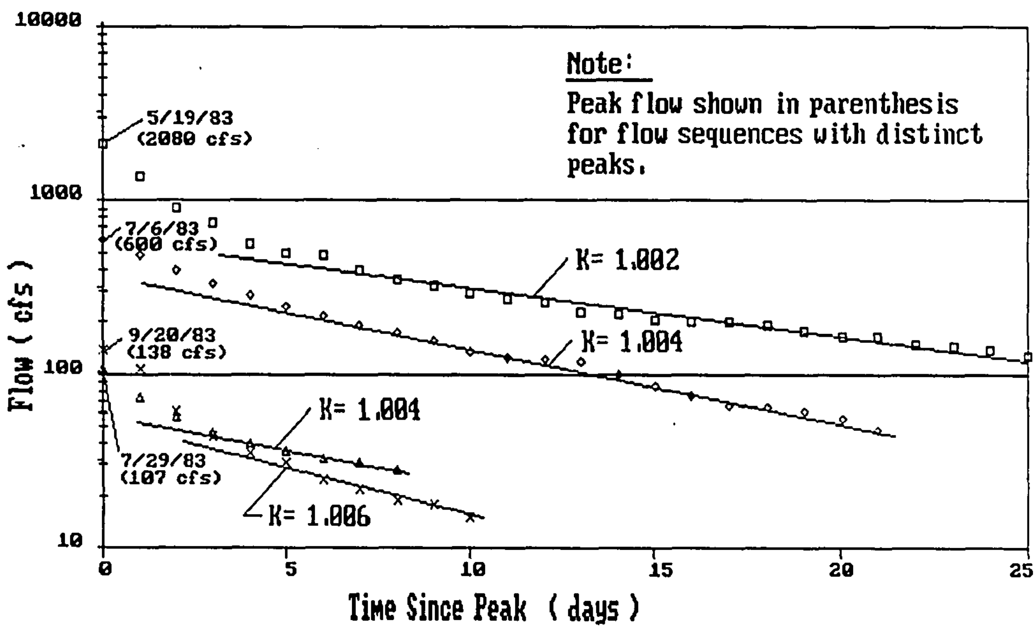

Figure 24. Baseflow recession on Squaw Creek in 1983

basins of 100 to 1000 square miles, K ranged from 1.001 to 1.003. Based on Howe's data and the recession flow analysis illustrated in Figures 23 and 24, K= 1.004 was selected for use during initial model calibration efforts. This value was subsequently increased as will be explained in discussion of the calibration process.

Howe also correlated q_0, the flow rate at which baseflow recession begins, with physical and climatic parameters. Observed values of q_0 during 170 low flow sequences yielded five regression equations--one for each month from May through September--similar to the following one for June.

$$q_0 = A^{1.01} T^{-1.12} I^{1.92}$$

where

q_0	=	The initial low flow discharge (cfs) at the beginning of a period of recession lasting at least 10 days;
A	=	Basin area (square miles);
T	=	Average daily temperature (degrees Fahrenheit) during a 10 to 20 day period preceding onset of baseflow;
I	=	Soil permeability index based on major soil associations in the basin.

Using an average temperature of 70 degrees, a value of 3.4 for I (Howe's recommendation for Clarion-Nicollet-Webster soil association), and a basin area of 227 square miles, the predicted q_0 for the Squaw Creek basin is only 21.6 cfs.

Clearly, this is much lower than most of the recession flow sequences shown in Figures 23 and 24. This discrepancy is due to the fact that Howe studied drought-like streamflow sequences since his goal was to predict onset of the protected low flows mandated by state law during drought periods. By contrast, no attempt was made here to analyze drought flow sequences. In fact, two of these recession sequences followed daily average flow peaks in excess of 1000 cfs (instantaneous flow peaks were even higher).

Since predicting high flows--rather than drought sequences--was the goal of this research, q_0 values of approximately 10 percent of peak flow was selected for initial calibration trials. This decision was based on observation of the flow sequences shown in Figures 23 and 24. Note that those hydrographs with distinct peaks (peak discharge shown in parenthesis) make a transition to a reduced recession rate at 10-40 percent of peak flow. Since superposition of subbasin hydrographs can inflate the value of q_0 for the total basin, as previously illustrated in Figure 22, setting q_0 at 10 percent of peak flow seemed a reasonable starting estimate for model calibration.

Hydrograph Routing HEC-1 offers two types of routing methods, hydraulic and hydrologic, for simulating changes in hydrograph shape as a flood wave moves along a channel.

The kinematic wave method is a hydraulic routing procedure based on the continuity equation and the momentum equation. According to the HEC-1 users manual, the kinematic wave model assumes simple uniform channel geometry and it does not provide for peak flow attenuation. For this reason, its use is most appropriate for urban storm sewers and channels where flood wave attenuation is not significant (U.S. Army Corps of Engineers 1985, p. 41).

Hydrologic routing methods are based on the continuity equation. HEC-1 offers several hydrologic routing methods including the Muskingham, Modified Puls, and Working R and D procedures. The Modified Puls Method--and the Working R and D method, a modification of the Modified Puls Method--are storage routing methods which can be used for either channel or reservoir routing. The HEC-1 users manual warns, however, that when used for channel routing, peak flow attenuation predicted by these methods is quite sensitive to the number of routing steps in each channel reach. These methods also rely on storage-outflow data derived from water surface profile studies. Since storage-outflow data were not available for the Squaw Creek channel, the Modified Puls and Working R and D methods were eliminated from consideration and the Muskingham method was adopted for use in the Squaw Creek Model.

The Muskingham routing method is based on the continuity equation and the following relationship between channel storage, inflow, and outflow:

$$S = K[xI + (1-x)O] \tag{15}$$

where:

- S = Channel storage (acre-feet);
- I = Average rate of inflow into a channel segment during a computational interval (cfs);
- O = Average rate of outflow from a channel segment during a computational interval (cfs);
- x = Muskingham weighting factor ranging from 0 to 0.5;
- K = Muskingham storage time constant approximating the travel time of the flood crest through the channel segment (hours).

According to Viessman et al. (1977, p. 233), the value of x for channels is typically about 0.2. This value was selected for initial calibration trials. Subsequent trials using values as low as 0.1 and as high as 0.3 caused less than five percent change in the predicted flood crest, so x was fixed at 0.2 for the Squaw Creek model.

Estimates of travel time through each of the six channel reaches shown in Figure 13 were based on channel lengths, scaled from topographic maps, and on calculated flow velocities. Channel geometry was approximated using cross sections from bridge design sheets for crossings at five locations in the basin. These were supplied by the Iowa Department of Transportation. Bankfull velocity was estimated using the Manning Equation. Average slope for each reach was obtained from topographic maps, and Manning's N factor was assumed equal to 0.03. Table 10 summarizes the velocity estimates. Based on these rough estimates, it was decided to use a value of 3.5 ft/sec in the upper part of the basin where the channel cross section is smallest, and values of 3.5 to 4.0 ft/sec in the lower part of the basin.

Table 10. Velocity estimates at Squaw Creek bridge crossings

Crossing Location	Average Slope (ft/ft)	Velocity Estimate (ft/sec)
Iowa Highway 175 Hamilton County	0.0006	4.2[a]
Iowa Highway 17 Hamilton County	0.0008	3.2
Boone-Story County Line	0.0007	3.3
County Road S 20 Franklin Twp. Story County	0.0010	4.1
Stange Road Ames	0.0005	3.9

[a]This cross section is a very deep drainage ditch near the basin divide. Bankfull flow is thought to be unlikely here and this velocity estimate is considered to be high.

Table 11 summarizes travel time estimates used in initial calibration trials for the Squaw Creek Model.

Table 11. Estimated channel travel time between subbasin nodes

Reach	Travel Distance (feet)	Estimated Velocity (ft/sec)	Travel Time (hours)
A - B	20,000	3.5	1.6
B - C	36,000	3.5	2.9
C - D	37,000	3.5	2.9
D - E	41,800	3.7	3.1
E - F	17,000	4.0	1.2
F - G	4,400	4.0	0.3

Model Calibration and Verification

After all submodels were selected and initial parameter estimates made, the Squaw Creek model was calibrated using precipitation and streamflow data for the largest flood of record, which occurred in June of 1975. Three other runoff events were used to verify performance of the calibrated model. The following sections of this report describe input data and procedures used to calibrate and verify the model.

Input Data

Streamflow Data Figures 25 and 26 are hydrographs for four runoff events used to calibrate and verify the Squaw

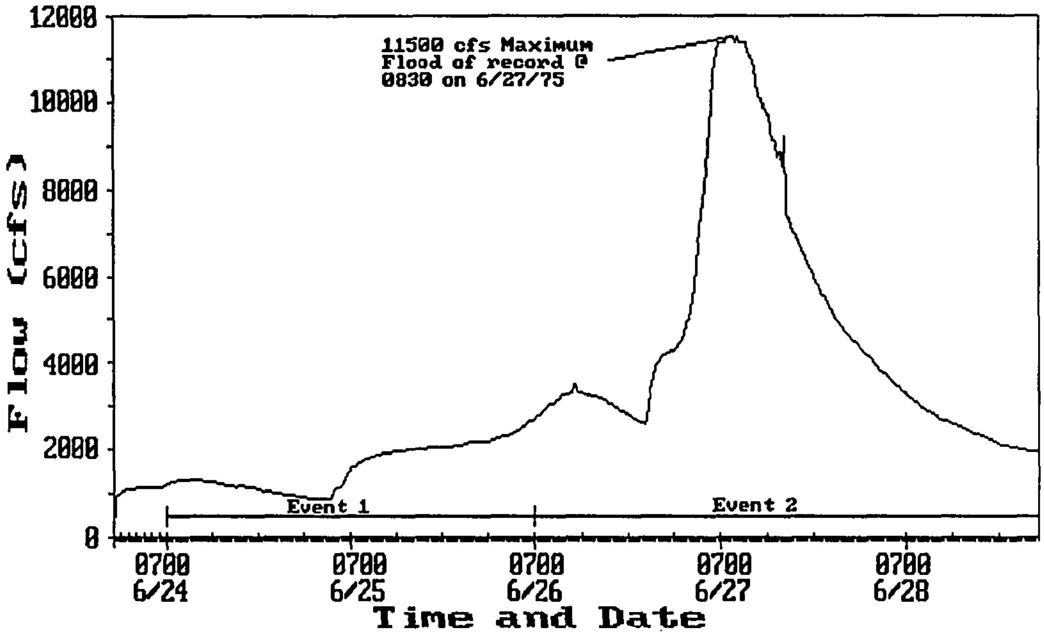

Figure 25. Hydrograph of Events 1 and 2

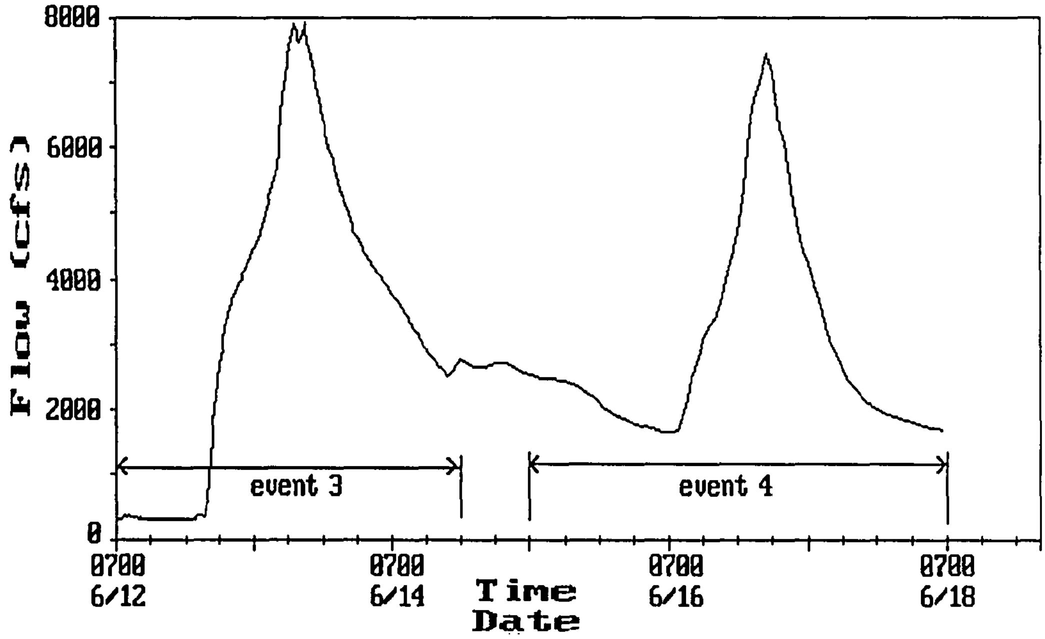

Figure 26. Hydrograph of Events 3 and 4

Creek model. Event 1 came in response to heavy localized rainfall in the northern part of the basin during the early morning of June 24th. The basin was moderately wet at the time of this rainfall (AMC II conditions for most subbasins) and streamflow response, as shown, consisted of a slow rise to an indistinct crest during June 25th. As shown in Figure 25 streamflow continued to rise slightly after Event 1. This is thought to be due to localized rainfall occurring in the Story City area the morning of June 26th.

Event 2, the most damaging flood to ever occur in the Squaw Creek basin, came in response to moderate basin-wide rainfall during the evening of June 26th. The basin was very wet from heavy rain on the morning of the 24th (Event 1), and Squaw Creek was running nearly bank full at Ames just prior to this event.

Event 3 occurred during June 12th and 13th of 1984 when very heavy basin-wide rains of two to four inches occurred around midnight of June 12th. The basin was moderately dry (AMC I conditions in all subbasins) prior to this event. Had the basin been wet, peak flow during this event might have exceeded that of Event 2.

Event 4 took place just two days after Event 3. It was caused by midday rains of one to two inches on June 16th that appeared to be centered over the western and central parts of the basin. Northern and southern areas of the basin received less than one inch.

All streamflow data were obtained through the Iowa City office of the U.S. Geological Survey. Considerable difficulty was encountered in obtaining records for the events occurring in 1975. These data had apparently been removed from active computer files and archived at a central data storage center. It took nearly six months to retrieve the 15-minute streamflow records. Had it not been for the diligent efforts of Mr. Oscar Lara in the Iowa City office of the U.S. Geological Survey, these records might never have been found.

The 1984 data were more easily obtained. Unfortunately, however, a malfunction in the automatic digital data recorder at the Squaw Creek gaging station caused most of the record after the Event 3 flood crest to be lost. Fortunately, members of the U.S. Geological Survey Field Office in Ft. Dodge, who maintain the Squaw Creek gage, took a number of manual weighted wire stage measurements during the last part of Event 3 and throughout Event 4. This permitted them to piece together a hydrograph for these events.

<u>Precipitation Data</u> Figures 27 - 34 show hourly and daily precipitation data associated with Events 1 - 4. These data were obtained from several sources. Five National Weather Service gages are located near the Squaw Creek basin but, as shown in Figure 35, only one is actually in the

basin. Three of the gages, those located at Webster City, Story City, and west of Ames, are recording stations which provide data for each 15-minute time interval. The other two gages, at Boone and Jewell, are read each day at 0700 and provide only 24-hour precipitation totals. In addition, the city of Ames maintains a gage at the Municipal Water Treatment Plant which is read every six hours.

To obtain an average storm precipitation for each subbasin, an areally weighted average of measurements from stations closest to the subbasin was calculated. Areal weighting factors for each subbasin were established by drawing a Theissen Net (Figure 35) and calculating the area of influence for each gaging station in every subbasin.

As shown in Figure 35, the relative locations of the six weather stations gave the Boone station dominant influence over precipitation estimates in the western part of the basin. This lead to a serious deficiency in runoff volume estimates during initial attempts to calibrate the model. While stations further to the east received three to four inches of precipitation during the three-day period when Events 1 and 2 took place, the Boone station recorded less than one-half inch. As a result, precipitation and runoff estimates for the western subbasins appeared to be substantially underestimated.

To overcome this problem, a supplemental source of

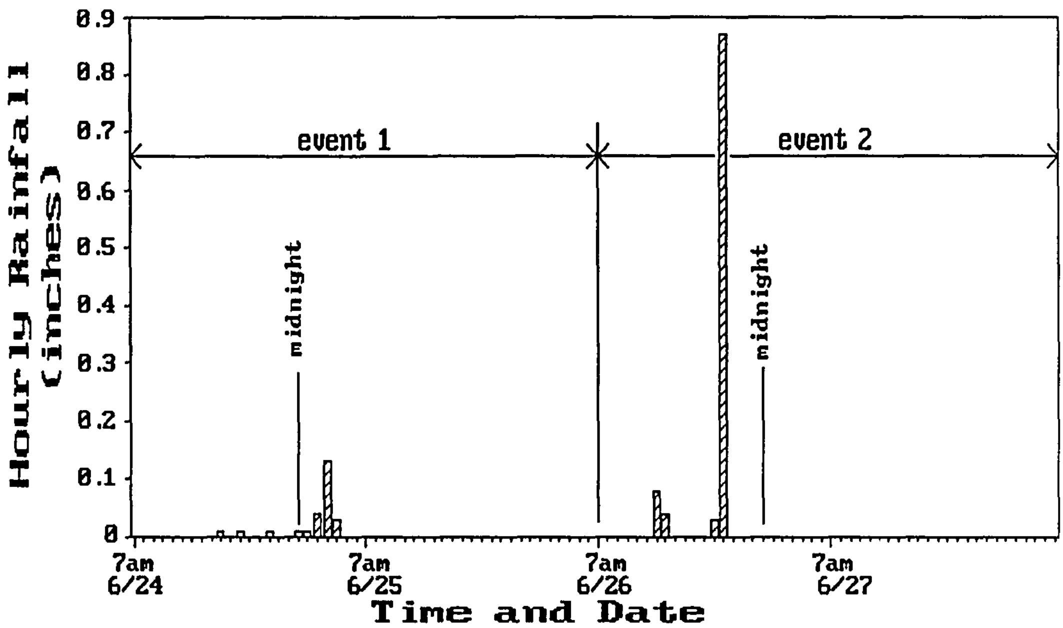

Figure 27. Hourly rainfall during Events 1 and 2 at weather station 8 miles west of Ames

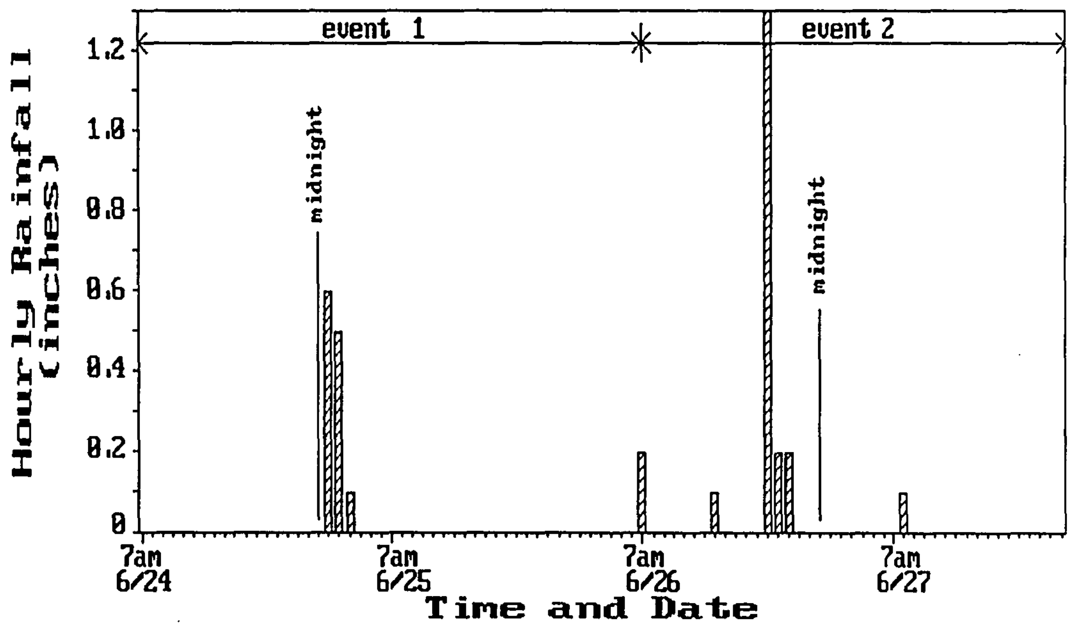

Figure 28. Hourly rainfall during Events 1 and 2 at weather station in Story City

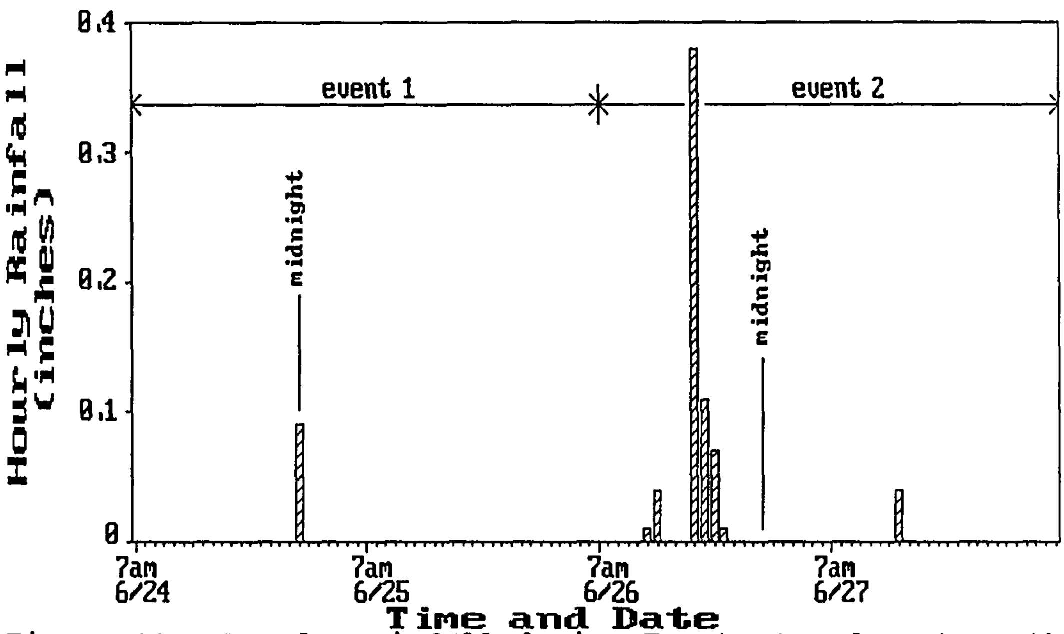

Figure 29. Hourly rainfall during Events 1 and 2 at weather station in Webster City

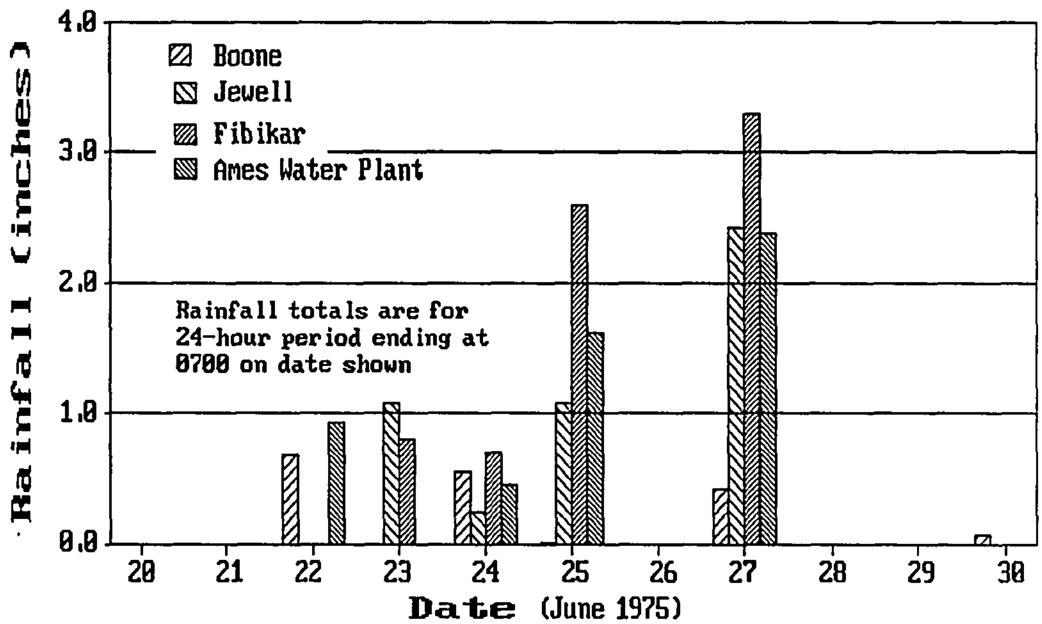

Figure 30. Daily rainfall prior to and during Events 1 and 2 at four weather stations

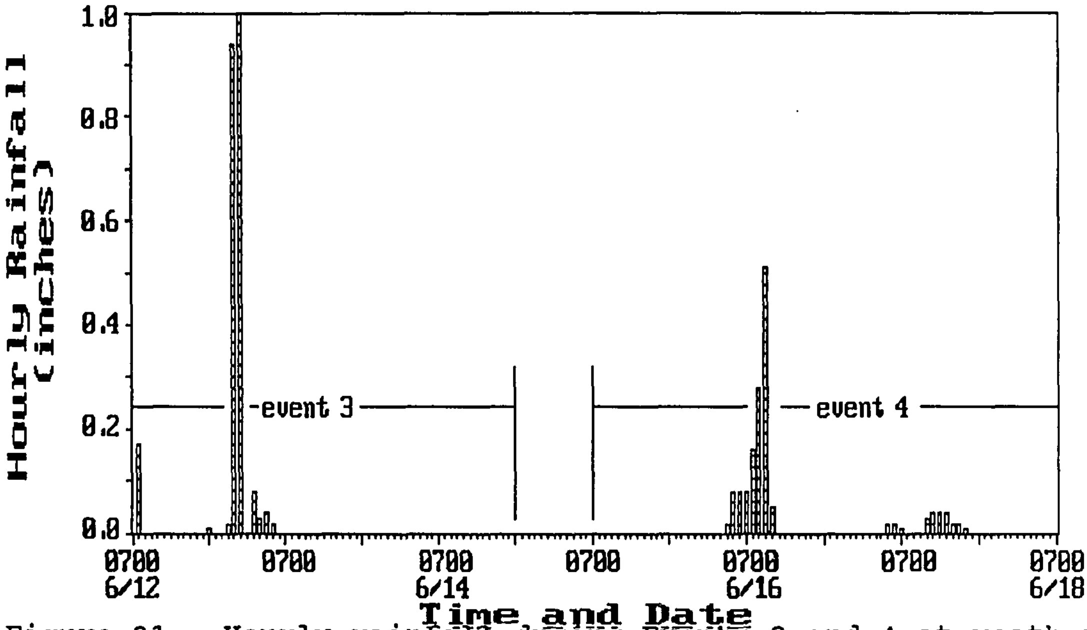

Figure 31. Hourly rainfall during Events 3 and 4 at weather station 8 miles west of Ames

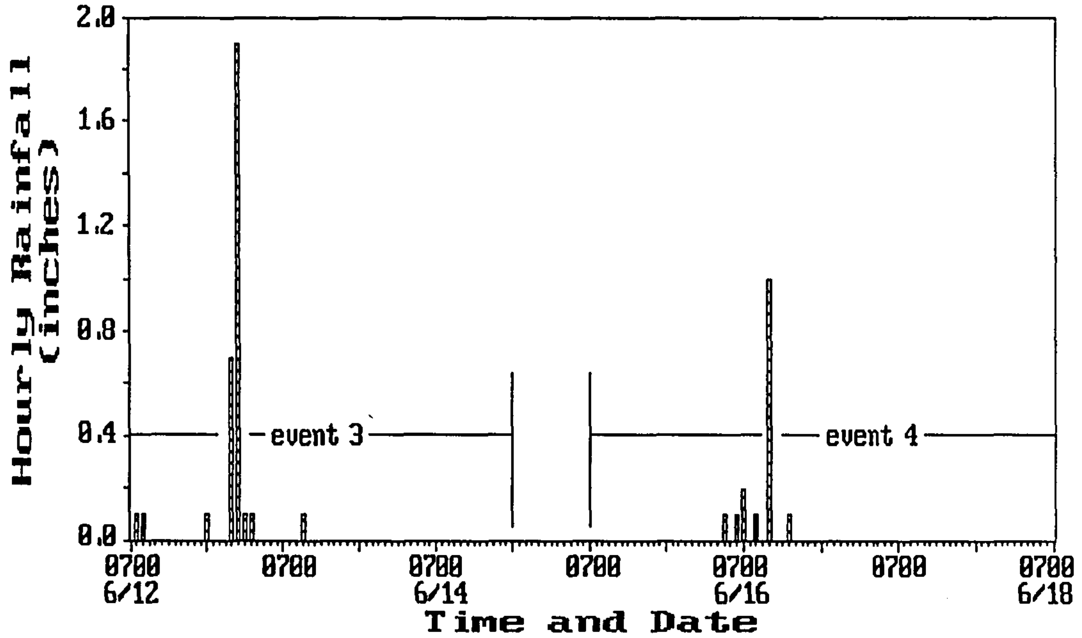

Figure 32. Hourly rainfall during Events 3 and 4 at weather station in Story City

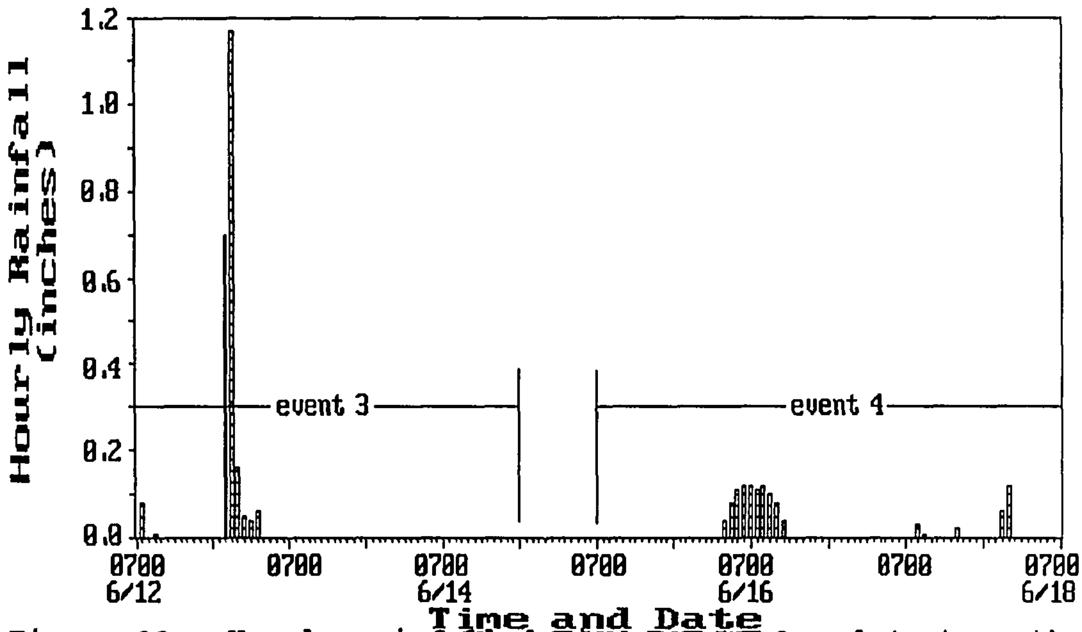

Figure 33. Hourly rainfall during Events 3 and 4 at weather station in Webster City

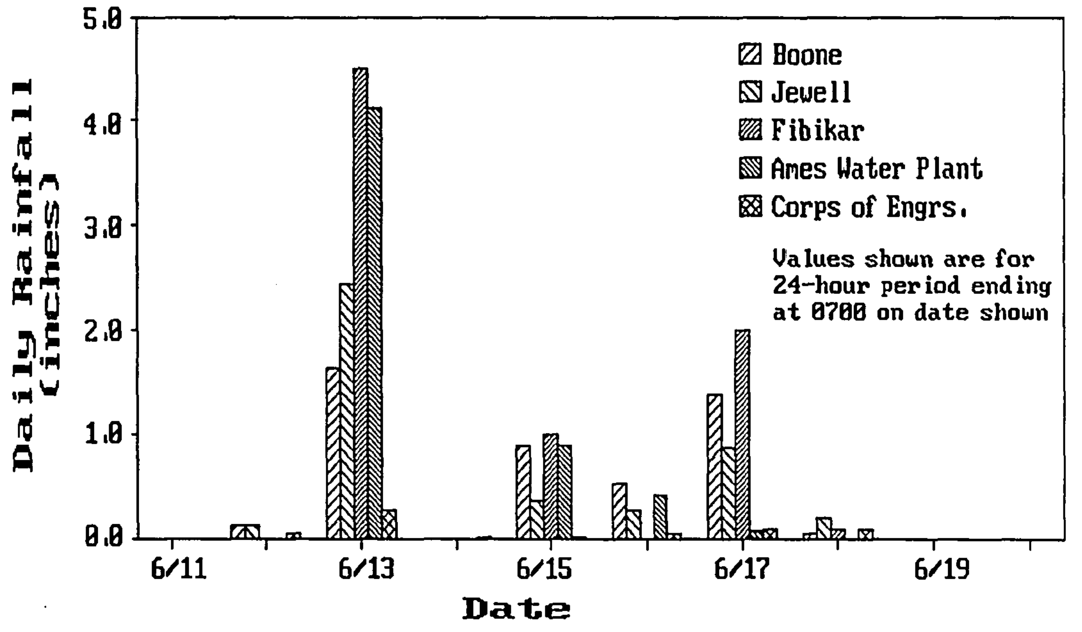

Figure 34. Daily rainfall during Events 3 and 4 at five weather stations

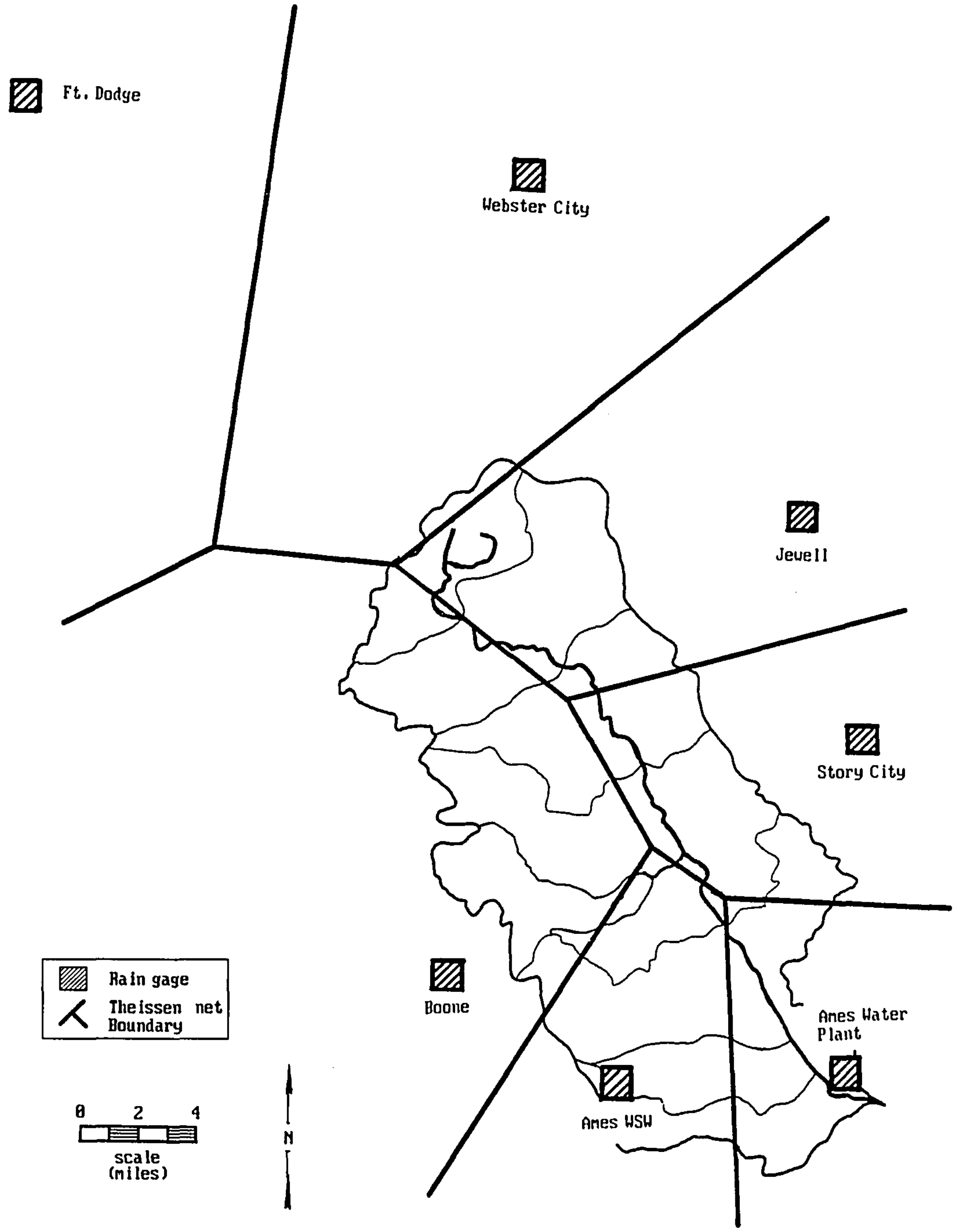

Figure 35. Theissen Net for municipal and National Weather Service precipitation stations

precipitation data was sought. In his unpublished observations of conditions prior to the 1975 flood, Dougal (1975) referenced rainfall records kept by Charles Fibikar, a farmer located in section 12 of Dodge Township in Boone County. Mr. Fibikar was contacted, and he agreed to share his records for the months of June 1975 and June 1984. A face-to-face interview with Mr. Fibikar revealed that he maintains several small precipitation gages on his farm and that he has kept daily precipitation records since 1952.

A comparison of Mr. Fibikar's data for June 1975 with National Weather Service data shows that his records coincide reasonably well with Weather Service measurements. On occasion the Fibikar data seem to be offset by one day from Weather Service reports. This seems to be due to the fact that Mr. Fibikar reads his gages at varying times during the day--sometimes just before going to bed, other times at breakfast--and that he may record a late evening or early morning rain as occurring on either of two possible dates.

Further attempts to locate supplemental rainfall data were unsuccessful. The Boone County Extension Director was able to supply names of farmers who keep seasonal rainfall records, but telephone contacts with these people revealed that none kept detailed records nor did they preserve them for more than a year.

A revised Theissen Net, incorporating the Fibikar gage, is shown in Figure 36. Clearly, subbasin rainfall estimates in a large portion of the western half of the basin are heavily reliant on the Fibikar data. This caused some difficulty in calibrating the model, as will be described later, but this situation was considerably better than basing basin rainfall estimates for the western subbasins solely on data from the Boone weather station which lies outside the basin.

An additional source of rainfall data was found for Events 3 and 4. Since 1980, the U.S. Army Corps of Engineers has maintained a meteorological station on the Des Moines River near Stratford, Iowa. This station was established to provide information used in operating the Saylorville and Red Rock Reservoirs.

Precipitation data supplied by the Corps for time periods coinciding with Events 3 and 4 was not used in the modeling effort, however, because it deviated substantially from data supplied by other weather stations in the area. As shown in Figure 34, the Corps rainfall data are much lower than data from surrounding stations. The next closest Weather Service station to the north and west, at Ft. Dodge, also disagreed significantly with the Corp's data.

The cause of this discrepancy is unknown. A visit to the Corp gage site showed it to be located on the bridge spanning the Des Moines River. There appeared to be no

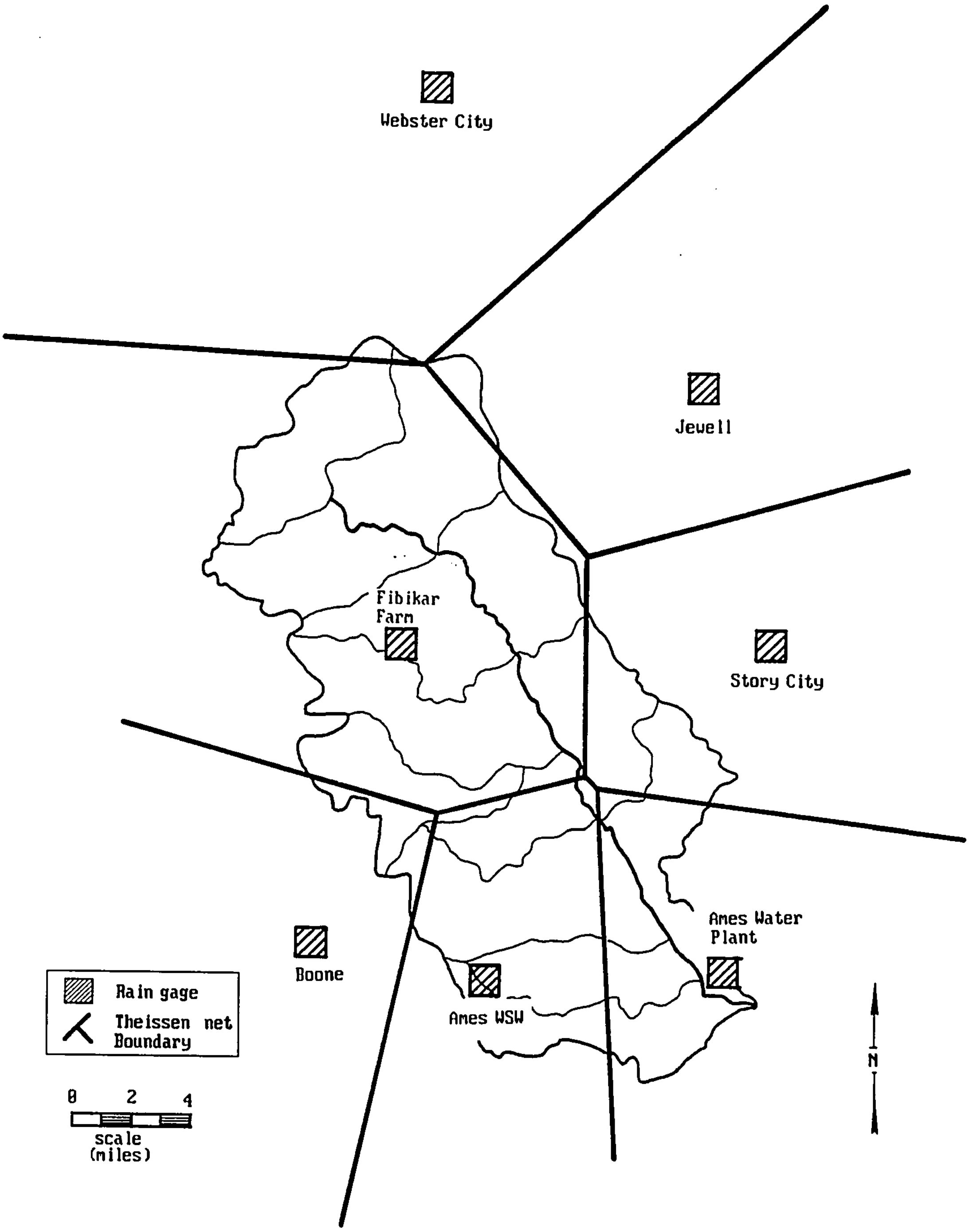

Figure 36. Revised Theissen Net using Fibikar gage

obstructions to shelter the gage other than the river valley itself. Regardless of the cause, the Corp precipitation data near Stratford were not felt to be representative of rainfall received in the northwestern part of the Squaw basin, and it was decided not to use these data to characterize Events 3 and 4.

Temporal, as well as spatial rainfall patterns, are important in basin modeling. HEC-1 allows the modeler to specify a hypothetical temporal pattern or to use temporal patterns from recording gages to establish the timing of rainfall throughout the basin. Data from National Weather Service recording gages at Webster City, Story City, and near Ames, were used to establish rainfall patterns during model calibration and verification. A Theissen Net (Figure 37) was drawn for these stations to establish areal weighting factors for each subbasin.

Procedures used to develop a temporal rainfall sequence for each subbasin are as follows:

1. Estimate P_T, an areally weighted average total subbasin storm precipitation based on data from <u>all</u> stations (recording or daily) whose Theissen Net boundaries intersect the subbasin;

2. Estimate P_R, the areally weighted average total subbasin storm precipitation at <u>recording</u> stations whose Theissen Net boundaries intersect the subbasin;

3. Estimate P_{Ri}, the areally weighted precipitation occurring during computational interval i at <u>recording</u> stations whose Theissen Net boundaries intersect the subbasin;

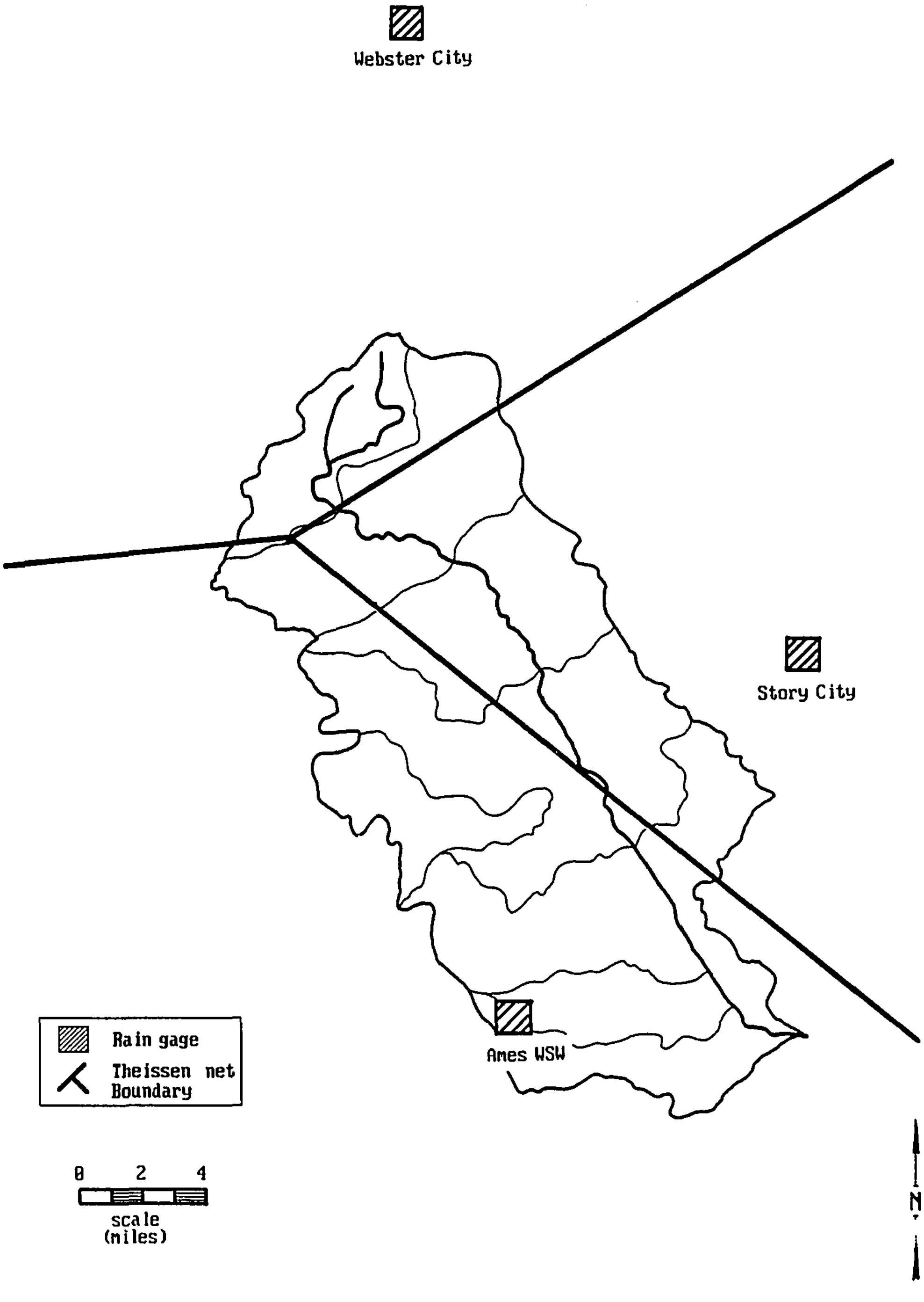

Figure 37. Theissen Net for weather stations used to establish temporal rainfall patterns

4. Calculate the fraction F, of P_R which fell during each computational interval ($F = P_{Ri}/P_R$); and

5. Calculate P_{Ti}, the estimated amount of precipitation occurring during computational interval i ($P_{Ti} = F*P_T$).

Data and Parameter Adjustment

During model calibration, several adjustments were made to parameters and input data to bring the predicted flood hydrograph into agreement with streamflow records for the calibration event.

Precipitation Data   Initial calibration runs yielded predicted hydrographs with both peak flow and total runoff volume well in excess of target values. This can be caused by over-estimating storm precipitation or by use of SCS runoff curve numbers that are too high.

To bring the predicted hydrograph closer to the target, it was apparent that either the SCS curve numbers or the storm precipitation for some subbasins would have to be reduced. Since the whole basin had been quite wet throughout the month of June, the AMC III conditions originally estimated for most of the subbasins seemed realistic. This meant that excessive runoff predicted by the model was due primarily to over-estimation of precipitation in one or more subbasins.

A review of precipitation at each rainfall station showed measurements at the Fibikar farm to be considerably higher than those at surrounding stations (Table 12).

Table 12. Rainfall totals for calibration event (6/26-27/75)

Precipitation Station	Total Precipitation (inches)
Webster City	0.62
Jewell	2.43
Story City	2.00
Fibikar Farm	3.30
Boone	0.42
Ames Water Plant	2.38
Ames 8WSW	1.02

As shown in Figure 36, the large Theissen cell around the Fibikar gage causes precipitation measurements at this location to heavily influence rainfall estimates throughout the northern part of the basin. As a result, heavy precipitation recorded at the Fibikar gage was attributed to a large area encompassing all or portions of several subbasins. Much lower rainfall recorded at Webster City, Boone, and Ames 8WSW suggested, however, that this heavy rainfall was very localized and did not accurately represent rainfall over a large area.

To reduce predicted runoff for the upland portion of the basin, a series of trial runs was made with the Fibikar data scaled down by increments of 0.5 inches. As shown in Figure 38, predicted peak flow and runoff volume came closest to equaling the historic record when the Fibikar precipitation totaled 1.8 inches. Since rainfall measurements at National Weather Service stations in Jewell and Story City (approximately 12 miles from the Fibikar farm) ranged from 2.0 to 2.4 inches, it was felt that rainfall in this range was more likely to be representative for the northern half of the basin than the much higher total reported at the Fibikar farm. For this reason, the precipitation at the Fibikar gage was artificially reduced to 2.3 inches. Subsequent lag time, routing, and baseflow parameter adjustments, which improved the overall shape and timing of the predicted hydrograph, further reduced the predicted crest below the target level. The Fibikar rainfall was readjusted upward to 2.5 inches at the end of the calibration process to correct this.

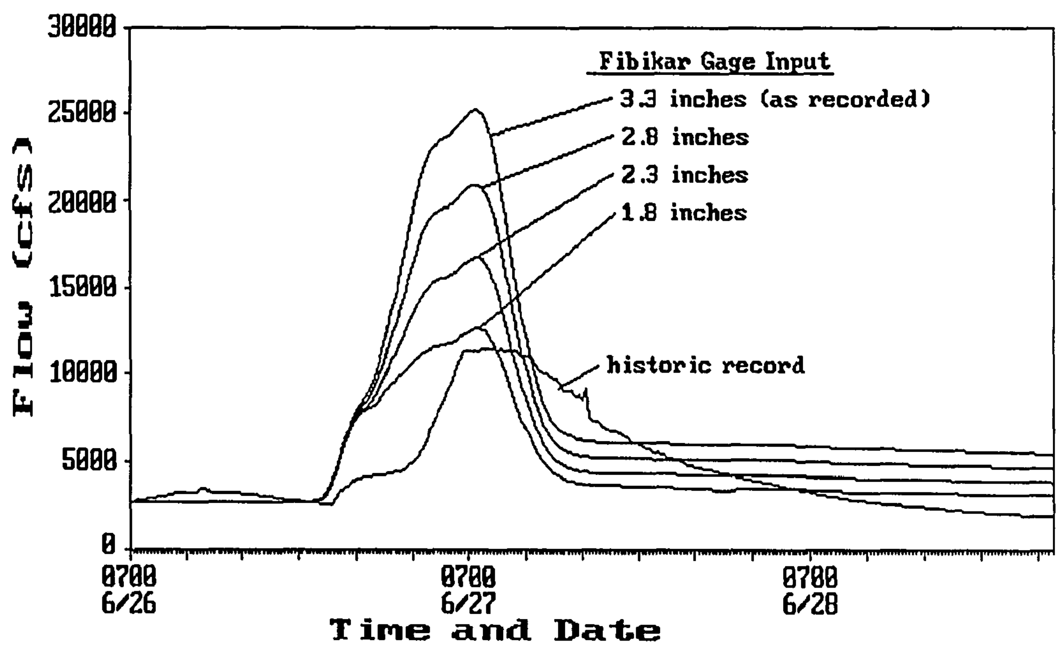

Figure 38. Model predictions during calibration for various levels of rainfall at Fibikar farm

Subbasin Lag Time   With rainfall estimates for northern subbasins revised downward, as just described, the projected peak was still high. As shown by equation 7, peak flow for an SCS unit hydrograph is inversely proportional to subbasin lag time. Clearly, an increase in subbasin lag times could improve the predicted hydrograph by reducing the peak.

A second possible remedy was to change the Muskingham routing parameters so as to attenuate the flood crest. Numerous trial runs demonstrated that either remedy, as well as combinations of the two, could force the model to mimic the historical record. Considerable time and effort was spent trying to justify one approach over the other.

This finally culminated with the discovery, as described earlier, that estimated subbasin times of concentration (and lag time) were systematically underestimated. Table 13 summarizes the original and revised lag times. Improvement in the predicted hydrograph resulting from use of the revised lag times can be seen in Figure 39.

Routing Parameters   Following implementation of the revised subbasin lag times, it was noted that the recession limbs of predicted hydrographs dropped off much too rapidly. To prolong the upper portion of the recession limb, it appeared desirable to delay arrival of flow components originating in subbasins furthest from the gaging station.

Table 13. Original and revised lag time estimates

Subbasin	Original Lag Estimate (hours)	Revised Lag Estimate (hours)
A	2.4	5.9
B1	2.0	7.9
B2	2.3	4.9
C1	2.2	2.9
C2	1.3	3.1
D1	4.0	9.5
D2	3.0	3.6
D3	2.2	5.0
D4	1.5	2.3
E1	3.7	4.7
E2	2.0	8.0
F	2.7	6.3

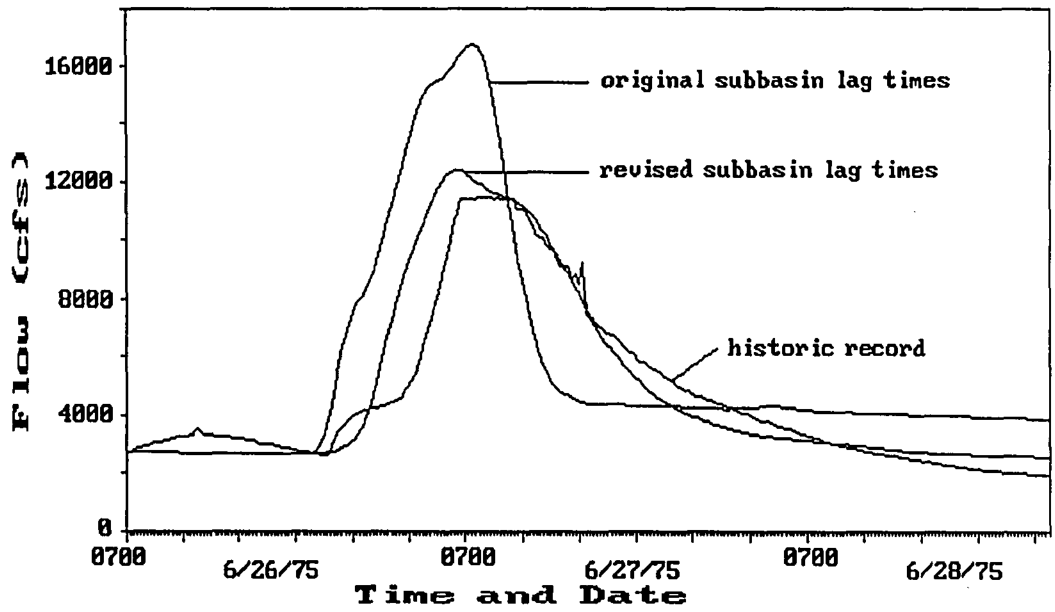

Figure 39. Predicted hydrograph using revised lag estimates

To obtain a clearer picture of how each subbasin contributes to flow at the gaging station, the HEC-1 model was restructured to individually route subbasin outflow hydrographs to the gaging station. Figure 40 illustrates output from the restructured model. For the purposes of illustration, the number of component hydrographs was reduced from 12 to 5 by combining flow from adjacent subbasins prior to routing. Thus, each of the five component hydrographs in Figures 40 and 41 represents the flow contribution from a cluster of two or more subbasins.

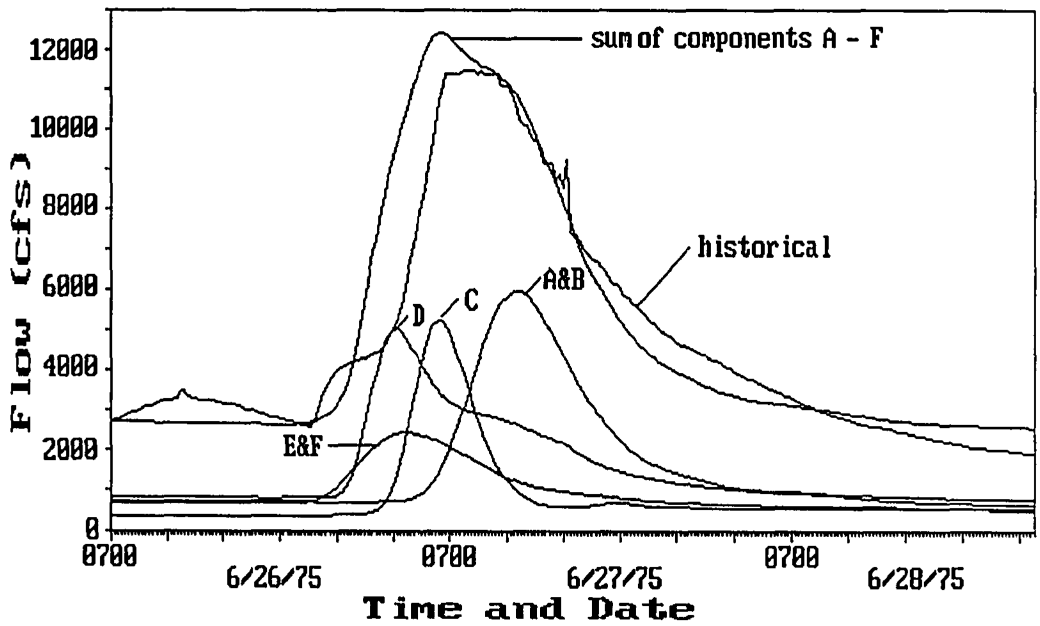

Figure 40. Subbasin contribution to total flow using original estimates of Muskingham routing parameters

As would be expected, subbasins nearest the gage contribute most heavily to the rising limb of the basin hydrograph, while upland subbasins contribute most of the flow beneath the falling limb. By extending the travel times for upstream reaches of the main channel, it was possible to slow the arrival of flow from the northern subbasins, thereby extending and flattening the falling limb of the predicted hydrograph. Figure 41 shows the impact of extending the travel time in reach D-E from approximately 3.1 hours to 5.3 hours. This delayed arrival of flow components from subbasins A,B,C, and D.

Implicit in this change is the assumption that either the total routing distance for each of these subbasins was originally underestimated, or that flow velocity in the main channel was overestimated. It is believed that both types of error may have been made in deriving the original routing parameters but that inaccurate measurement of the total flow distance was the more significant. Since the main channel is quite winding in the upper part of the basin, and flow distances were crudely measured from topographic maps at a scale of 1:24000, flow distances could have easily been underestimated.

<u>Baseflow Parameters</u> The final calibration adjustments made were to K and q_0, parameters that control baseflow recession according to equation 13. Figure 41 shows

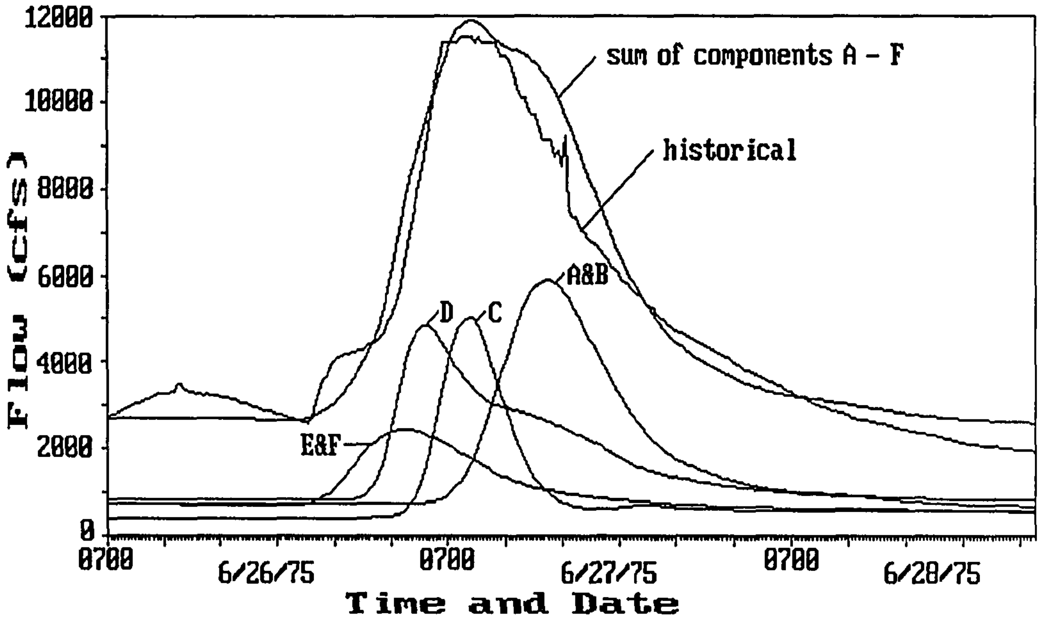

Figure 41. Predicted hydrographs after extending travel time for subbasins A - D

the tail of the predicted hydrograph to be much too flat, indicating that K (originally estimated at 1.004) is too small. Trials were made with K ranging from 1.01 to 1.03 to find the value that causes the trailing tail of the predicted hydrograph to decline at a rate parallel to the historical record. Based on results of these trials, as shown in Figures 42 - 44, it appears that K = 1.02 yields a predicted recession that most closely parallels the historical record.

With K fixed at 1.02--and q_0 at 10 percent of subbasin peak flow as originally estimated--the trailing portion of the recession limb was below the target sequence of flows (Figure 43). To correct this, additional trials were made with q_0 at 20 and 30 percent of subbasin peak flow. As shown in Figure 46, baseflow initiation at 30 percent of subbasin peak flow caused the flat portion of the predicted hydrograph to be parallel to, but significantly higher than, the target flow sequence. Initiating baseflow at 20 percent of subbasin peak flow caused the trailing segment of the falling limb to lie just slightly above the historical record, as shown in Figure 45, and it was decided to fix q_0 at this level.

As discussed earlier, Howe's work (1968) indicates typical baseflow recession constants for small basins are in the range of 1.002 to 1.006. Furthermore, baseflow recession during dry weather would be expected to start at flows below 100 cfs. Having selected recession parameters that are significantly larger than this, a rational explanation for this discrepancy was sought.

In their brief review of recession analysis, Linsley, Kohler, and Paulhus (1949, p. 153) show that the falling limb of a hydrograph may be separated into three distinct phases using semilog plots similar to Figure 47. Streamflow contributed by groundwater plots along a straight-line at the lower end of the falling limb. If interflow plays a significant role in streamflow, a second straight-line segment, with a steeper slope than the groundwater recession line, will be observed higher on the falling limb. Finally,

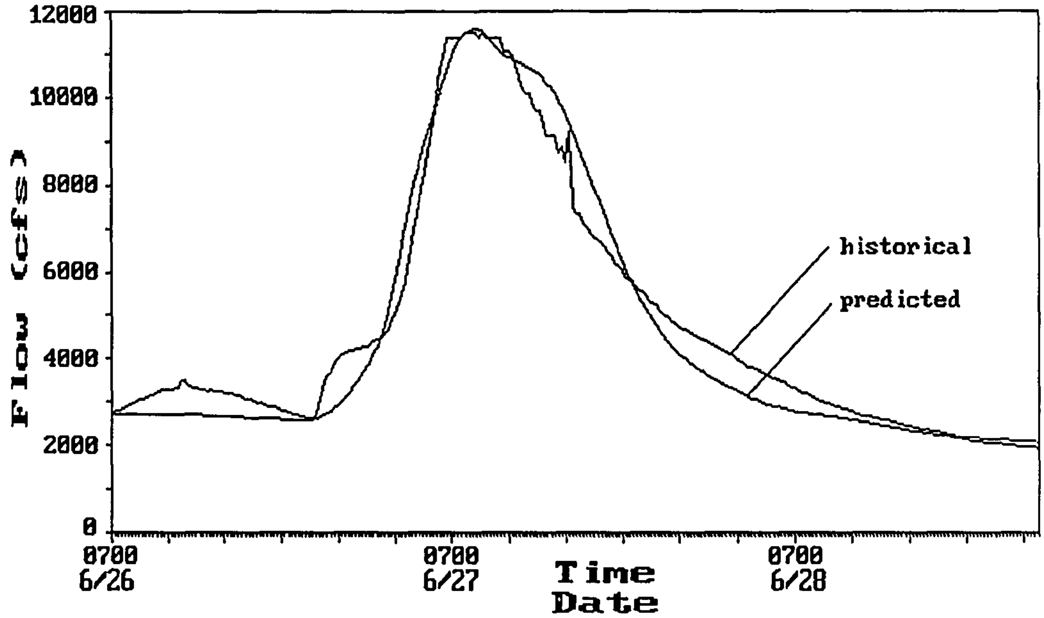

Figure 42. Predicted hydrograph with K=1.01

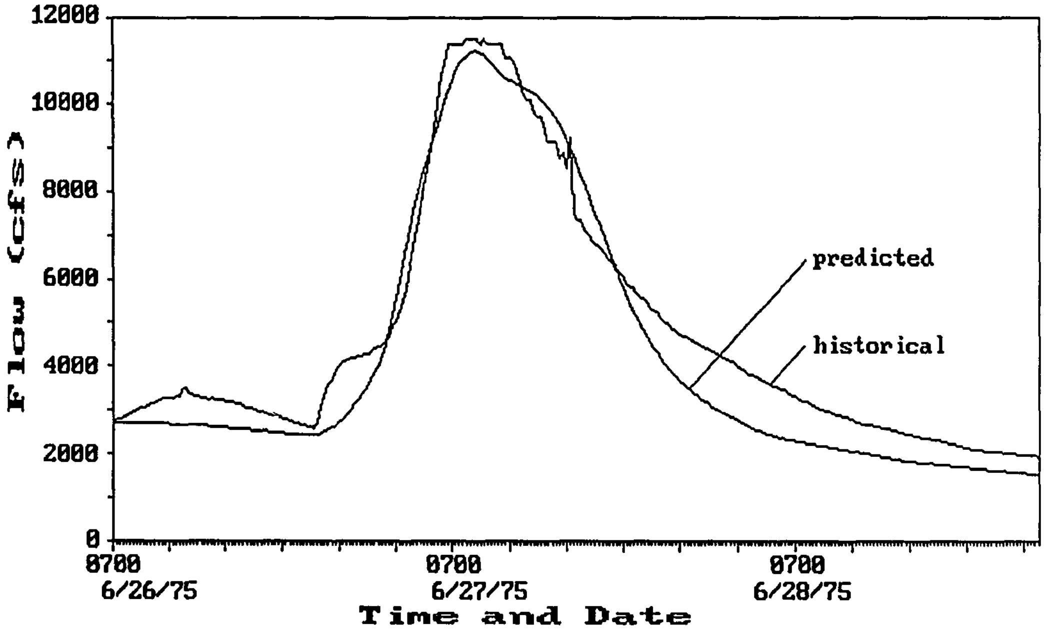

Figure 43. Predicted hydrograph with K=1.02

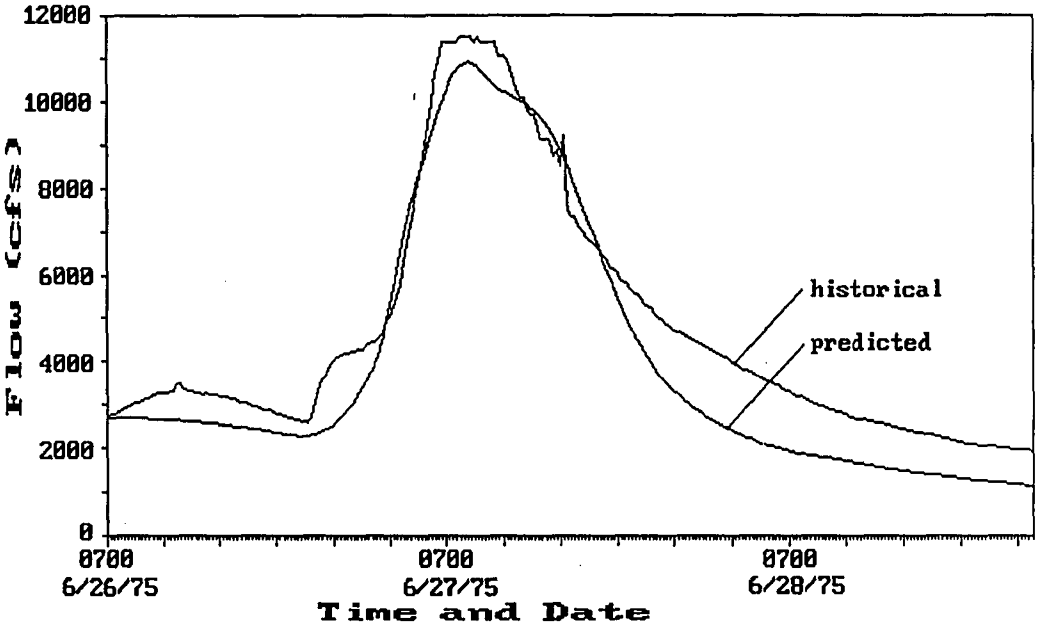

Figure 44. Predicted hydrograph with K=1.03

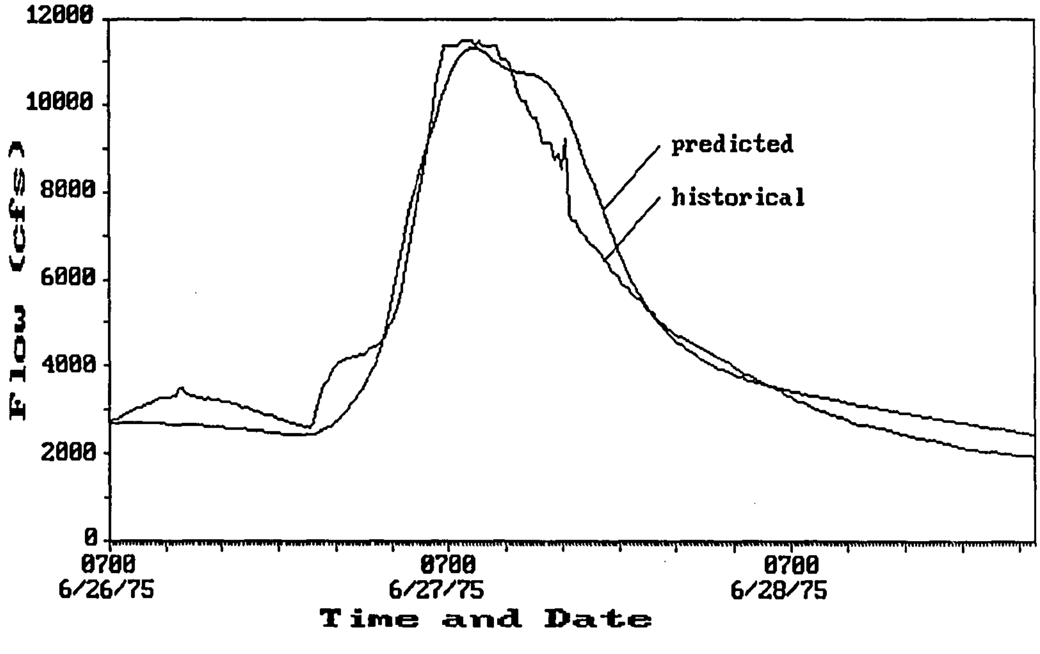

Figure 45. Predicted hydrograph with q_0 at 20 percent of subbasin peak flow

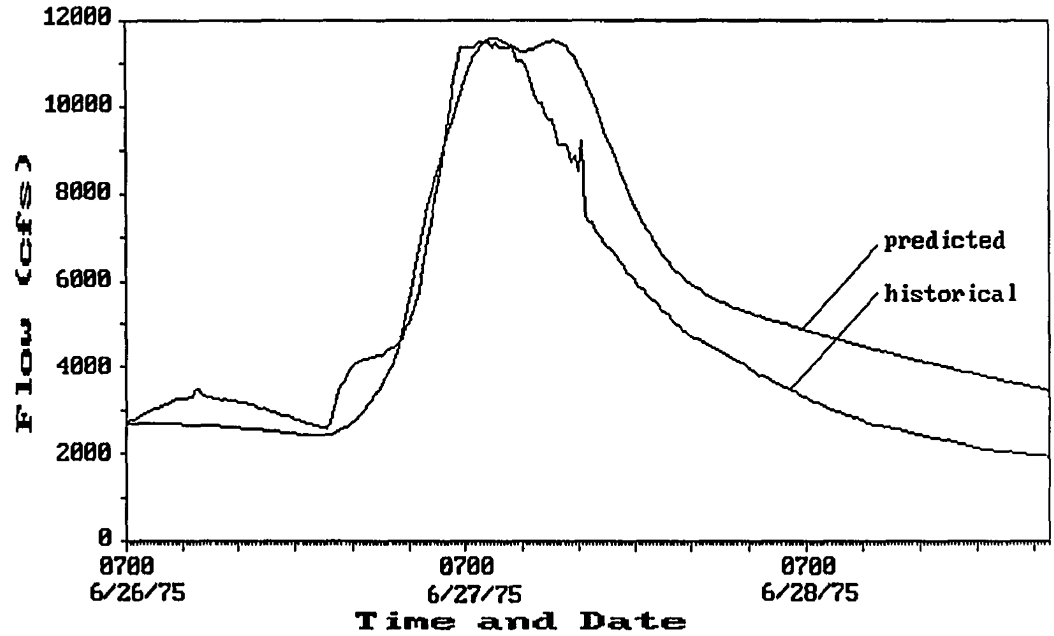

Figure 46. Predicted hydrograph with Q_0 at 30 percent of subbasin peak flow

near the top of the falling limb, a third straight-line segment may be observed which is representative of direct surface runoff.

Since no streamflow data are available for any of the Squaw Creek subbasins, streamflow for the total basin was plotted to see if a significant interflow component is present. Figure 47 shows daily average flow that occurred following the historic flood of 1975. The graphical analysis of daily average flow shows a clear indication of an interflow component with a recession constant of approximately 1.01.

The likelihood of a significant interflow component in

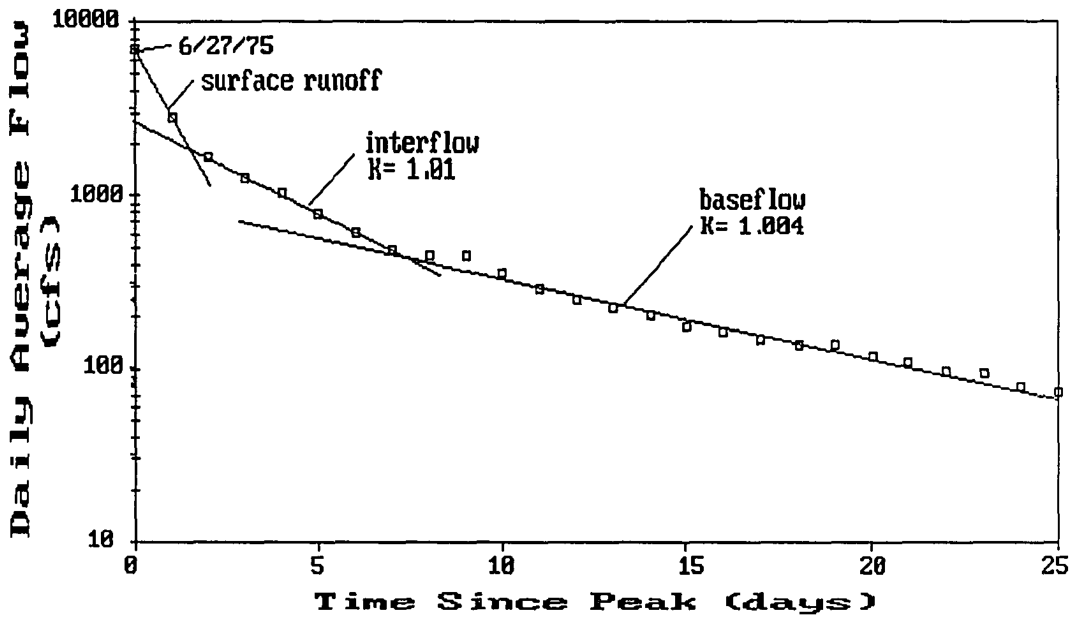

Figure 47. Recession analysis for flood of June 1975 on Squaw Creek

the Squaw Creek basin is further supported by studies of the effects of artificial drainage of agricultural land on downstream flooding.

In his study of flood hydrology in depressional watersheds, DeBoer (1969, p. 89) reported that watershed models predict moderate increases in peak flow as subsurface drainage is added to agricultural basins. The projected recession curve for basins with subsurface drainage is also somewhat flatter than for a basin with no artificial drainage.

When artificial drainage consisted primarily of surface drainage, projected peak flow increased substantially and the recession curve became steep. A combination of surface and subsurface drainage produced equally high peak flow, but the falling limb was projected to be less steep than for watersheds drained only by surface drainage.

These results suggest that subsurface drains deliver water to the receiving stream more slowly than surface runoff, but more rapidly than groundwater flow. As a result, basins like Squaw Creek that are extensively drained by subsurface tile, are likely to exhibit an interflow component.

Similarly, in their review of papers dealing with the effects of land drainage on streamflow, Irwin and Whitely (1983) emphasize that tile drainage provides temporary storage capacity in the soil profile by lowering the water table. Again, the result is more rapid movement of water to the receiving stream than provided by groundwater flow, but slower delivery than is provided by surface runoff. The result is a flow component that mimics interflow.

Model Verification

Event 3 was selected as the first verification trial using the calibrated model. It was selected because it was a high flow event--similar to the calibration event--and a reasonably complete hydrograph was available to document the peak flow and its time of arrival. The only missing streamflow data for this event were in the recession portion of the hydrograph, which was lost due to failure of the data recording system. Missing data were supplemented with manual readings taken by U.S. Geological Survey personnel at the time of the flood.

Precipitation data was input to the model without adjustment. SCS curve numbers were determined according to weighted five-day precipitation totals for each subbasin. The results of this verification run are shown in Figure 48 which shows the predicted and observed peak flows to be within 200 cubic feet per second of each other. Although the crest of the predicted hydrograph is somewhat broader than that of the observed, its timing is good. The total volume beneath the predicted hydrograph is about 16 percent less than that of the historical hydrograph.

Figure 49 shows results of the verification run for Event 4. Using the 2.00-inch rainfall reported at the Fibikar farm, the predicted peak is about 1700 cubic feet per second high, and the predicted runoff volume is about 28 percent greater than recorded. The crest also occurs approximately two hours early.

Since rainfall at nearby gages in Boone, Jewell, and Story City reportedly ranged from 0.90 to 1.38 inches, it is possible that, as in the case of the calibration event, the heavy rainfall at the Fibikar gage was not representative for the large land area bounded by the Theissen polygon

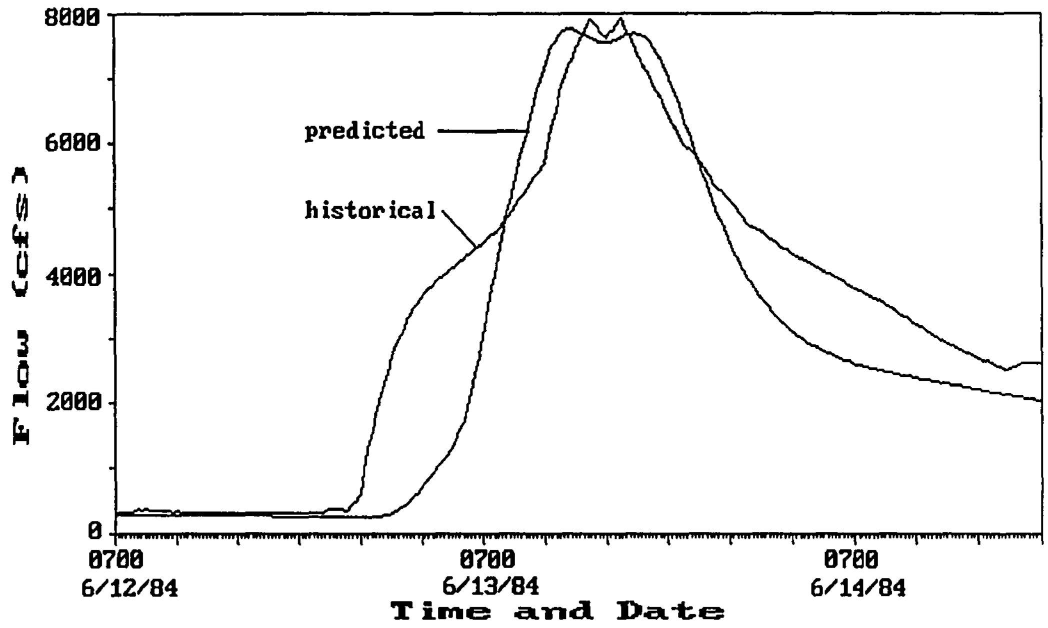

Figure 48. Predicted hydrograph for Event 3

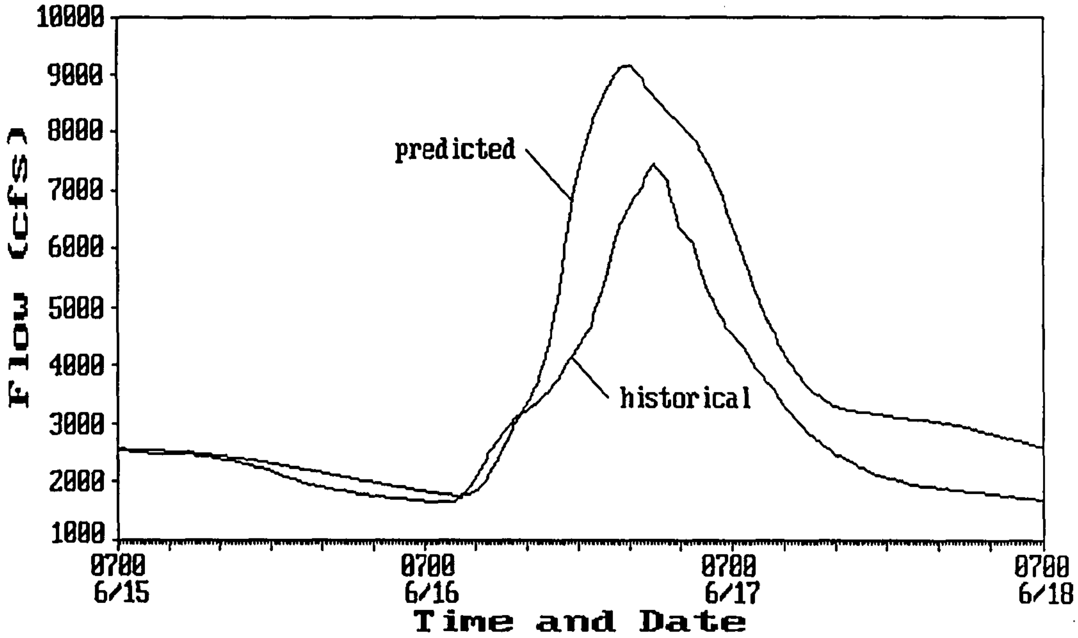

Figure 49. Predicted hydrograph for Event 4

surrounding this gage. Since Event 4 occurred when the basin was quite wet (AMC III conditions), predicted peak flow is quite sensitive to rainfall at the Fibikar gage. Reducing the Fibikar rainfall by only 15 percent (bringing it to 1.7 inches) results in a predicted peak flow that is within 300 cubic feet per second of the historical value. Since the historical hydrograph for Event 4 was derived from a relatively small number of manual weighted-wire stage readings--the automatic data recorder malfunctioned throughout Event 4--the historical peak and its time of occurrence are only approximate. For this reason, the agreement between predicted and observed hydrographs is felt to be reasonable.

Results for Event 1 are shown in Figure 50. This is the poorest of the three verification runs. Predicted peak flow is approximately 4000 cubic feet per second higher than recorded. Furthermore, because the heavy rainfall that caused it was apparently quite localized, the overall basin response was not large, nor was a distinct crest observed. As in the case of Event 2 (the calibration event), much of the difficulty with Event 1 is thought to be due to insufficient rainfall data and the fact that the Fibikar gage received much higher rainfall than any of the surrounding gages. As pointed out earlier, the model is quite sensitive to the Fibikar data since rainfall estimates for many of the

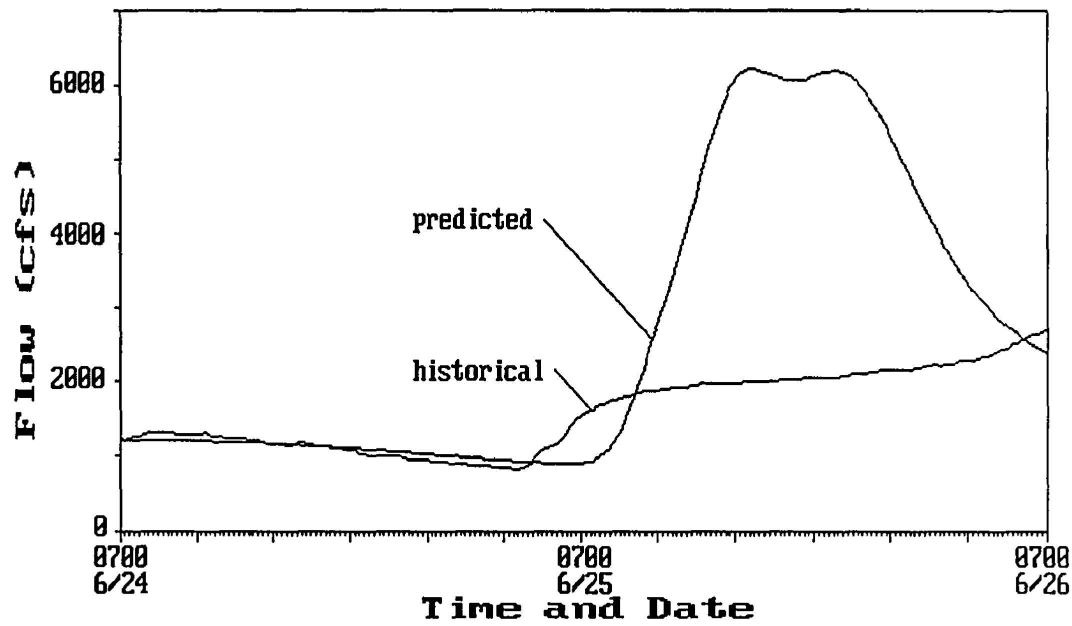

Figure 50. Predicted hydrograph for Event 1 using 2.6 inch rainfall at Fibikar station

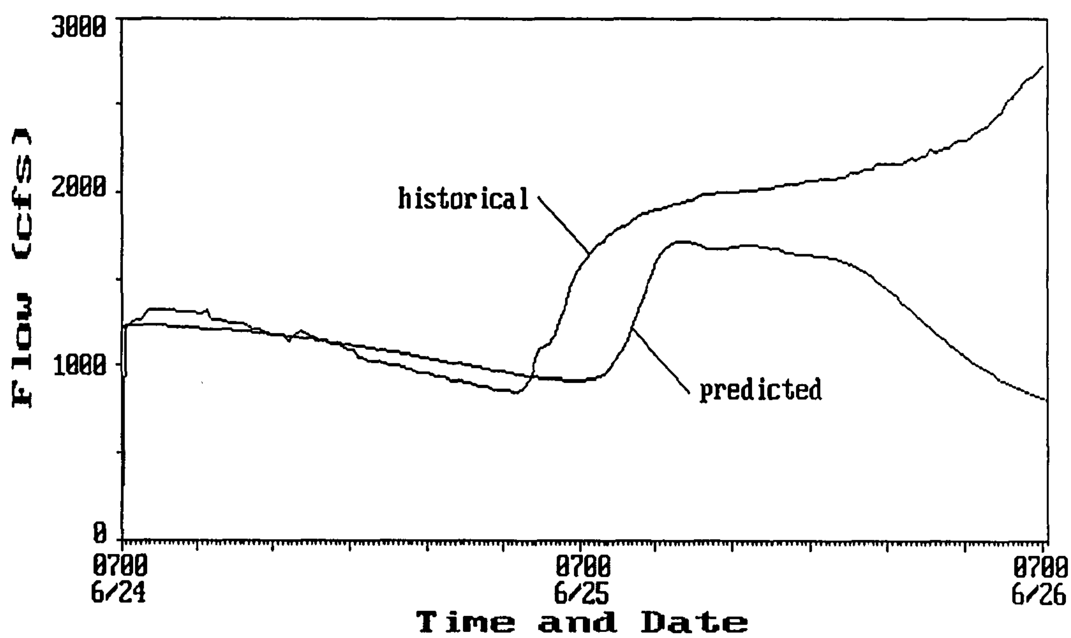

Figure 51. Predicted hydrograph for Event 1 using 1.2 inch precipitation at Fibikar station

northern and western subbasins are based on it. If the Fibikar precipitation for Event 1 is reduced from the reported 2.6 inches to 1.2 inches--the largest amount reported by any of the nearest gages--the predicted hydrograph for Event 1 looks like that shown in Figure 51.

Basin Response Modeling

Following calibration and verification the Squaw Basin Model was used to study basin response to changes in storm precipitation, duration, and direction of travel. The following sections summarize results that were felt to be significant in designing a simplified flood prediction procedure for possible use by the city of Ames.

Storm Quantity

One of the most obvious requirements for accurate flood prediction is to know approximately how much precipitation is necessary to cause flooding. To explore this relationship, a series of model runs was made with varying amounts of basin-wide rainfall.

As noted earlier, HEC-1 allows the modeler to input precipitation data in two ways. Rainfall data may be specified and weighting factors applied to estimate subbasin average rainfall or total rainfall, and a temporal pattern may be specified for each subbasin. The second alternative was employed for basin response studies so that exact precipitation amounts and durations could be specified for each subbasin.

To determine a reasonable upper range of input values for basin-wide precipitation, the Rainfall Frequency Atlas of the United States (U.S. Dept. of Commerce 1961) was consulted to determine the 25-, 50-, and 100-year return period storms for central Iowa. Table 14 summarizes these events.

Since it can significantly affect the peak flow forecast, baseflow prior to the hypothetical storms was set to zero so that the predicted hydrographs reflect only direct surface runoff. Typical model predictions are illustrated in Figure 52 which shows results for several basin-wide storms distributed uniformly throughout a three hour period. In this case, antecedent moisture condition II was assumed prior to the storm. Predicted peak flows range from 2000 cfs for one and one-half inches of rainfall, to 18000 cfs for a four inch event. Note that the peak occurs approximately 14 hours after the storms begin.

Table 15 summarizes predicted basin response to varying amounts of rainfall. One-inch rainfall increments and five different storm durations were used. The maximum rainfall used for each duration was selected so as to not exceed the 100-year storm by more than one inch.

For AMC I conditions, basin-wide precipitation of three inches (50-year return period for three-hour duration) would

Table 14. Rainfall depth (in inches) for several storm durations and frequencies in central Iowa

Duration (hours)	Return Period (years) 25	50	100
1	1.8[a]	2.0	2.2
3	2.7	3.0	3.4
6	3.4	3.8	4.2
12	4.2	4.8	5.3
24	5.0	5.5	6.1

[a]Values shown are point rainfall amounts that have been adjusted for a basin area of approximately 200 square miles.

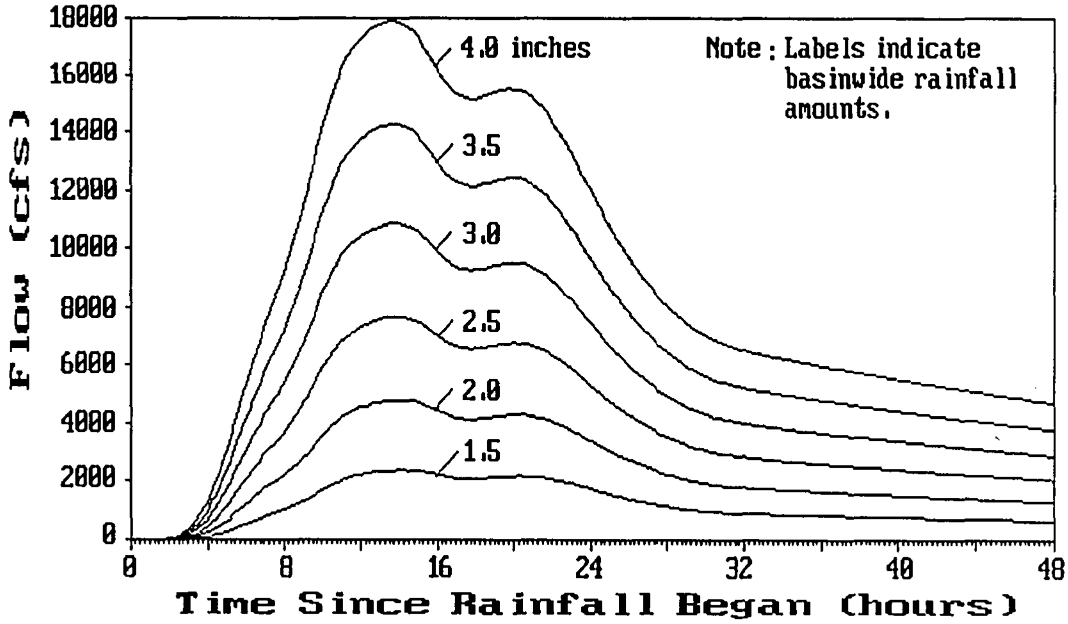

Figure 52. Predicted hydrographs for three-hour duration storms of various magnitude

Table 15. Peak flow predictions for Squaw Creek under various storm and antecedent moisture conditions

Antecedent Moisture Condition	Storm Duration (hours)	Rainfall Amount (inches) 2	3	4	5	6
I	1	800	3600	[a]		
	3	800	3600	7800		
	6	800	3500	7600		
	12	800	3400	7300	11900	17200
	24	700	3100	6500	10400	14700
II	1	4900	11100			
	3	4800	10900	17900		
	6	4700	10600	17400		
	12	4400	9900	16000	22600	29500
	24	3900	8300	13200	18400	23700
III	1	10600	18900			
	3	10400	18500	27000		
	6	10000	17900	26000		
	12	9200	16300	23500	30900	38300
	24	7500	12900	18400	23900	29400

[a]blank cells indicate precipitation values that are more than one inch greater than the 100-year storm.

be expected to cause minor flooding with a peak flow of approximately 3600 cubic feet per second. A summary rating table (Table 16) for the Squaw Creek gage shows that this flow would occur at approximately three feet above the designated flood stage of seven feet.

Under AMC II conditions, basin-wide rainfall of three inches (50-year return period for three-hour duration) is predicted to produce peak flows nearly equal to the historic flood of June 1975, and three to four inches of precipitation--a 100-year storm for durations of three to six hours--could easily cause record flooding.

Only two inches of precipitation--a 25-year return period event--is predicted to cause near-record floods under very wet (AMC III) conditions.

Rainfall Variability

While the data in Table 15 indicate response to uniform basin-wide precipitation, they tell little about the possible effects of heavy localized rainfall. To better understand the flooding potential of localized or spatially varied storms, a series of model runs was conducted in which

Table 16. Summary stage-discharge table for Squaw Creek gage at Ames, Iowa

Gage Height (feet)	Flow Rate (cubic feet/second)
3.0	530
4.0	1000
5.0	1390
6.0	1730
7.0	2040
8.0	2460
9.0	2930
10.0	3690
10.5	4100
11.0	4540
11.5	5220
12.0	6150
12.5	7190
13.0	8350
13.5	9650
14.0	11090

rainfall was restricted to a selected subbasin or cluster of subbasins. Antecedent moisture conditions II and III were used, and subbasin outflow hydrographs were routed to the Squaw Creek gaging station so that flooding potential in Ames could be determined. As the data in Table 17 show, the likelihood of flooding caused by runoff from any single subbasin is low. But, assuming AMC II conditions, rainfall of 3.5 to 4.0 inches over two or more adjacent subbasins is predicted to cause flow that would exceed the 7.0 foot designated flood stage by three to six feet. In the case of AMC III conditions, the same storms could easily cause significant flooding, particularly if streamflow is high prior to the storm event.

Storm Duration

As indicated by the summary results in Table 15, storms with equal runoff--those with the same total precipitation and AMC conditions--exhibited surprisingly little variation in predicted peak flow for durations of one to six hours. This was not originally anticipated since it was assumed that as storm duration increased the baseline of the runoff hydrograph would lengthen accordingly. Since the total amount of runoff was expected to stay constant for a given amount of rainfall, a substantial decline in peak flow was anticipated to offset the longer hydrograph baseline.

A closer look at individual subbasin hydrographs for

Table 17. Predicted peak flow from localized storms over one or more subbasins

Antecedent Moisture Condition	Subbasin(s)	Rainfall Amount (inches)				
		2.0	2.5	3.0	3.5	4.0
	A	789[a]	1228	1709	2221	2754
	B1,2	1606	2500	3478	4518	5603
	C1,2	1931	3076	4342	5708	7141
II	D1,2,3,4	2607	4131	5812	7624	9514
	E1,2	1603	2591	3697	4888	6143
	F	476	785	1133	1511	1912
	A	1602	2191	2794	3406	4024
	B1,2	3260	4457	5684	6929	8186
	C1,2	4088	5664	7287	8940	10624
III	D1,2,3,4	5502	7588	9743	11935	14153
	E1,2	3572	4975	6429	7913	9419
	F	1089	1531	1996	2474	2961

[a]Flow predictions assume zero streamflow prior to the storm event.

storms of varying duration reveals, however, that baseline length is not affected by storm duration as much as originally expected. This is due, mainly, to delayed initiation of runoff as storm length is increased. When duration of a fixed-volume storm of uniform intensity is increased, the rainfall rate declines. But, as storm intensity is reduced, a greater amount of time is required to satisfy initial rainfall abstractions that occur before runoff is initiated.

Figure 53 illustrates this with runoff hydrographs for subbasin D3 resulting from a three-inch storm of five different durations. In this case all baseflow prior to and after the crest has been removed to more clearly show the beginning and ending points of the hydrographs. Both the one- and three-hour storms produce hydrographs with baseline lengths (the elapsed time from initiation to completion of runoff) of about 22 hours. The six-hour event has a slightly longer baseline of approximately 23 hours. Baselines for the 12- and 24-hour storms are 26 and 34 hours respectively.

As anticipated, increasing the duration of a particular amount of basin-wide rainfall delays the arrival of peak flow at Ames. Table 18 recaps the effects of changing storm

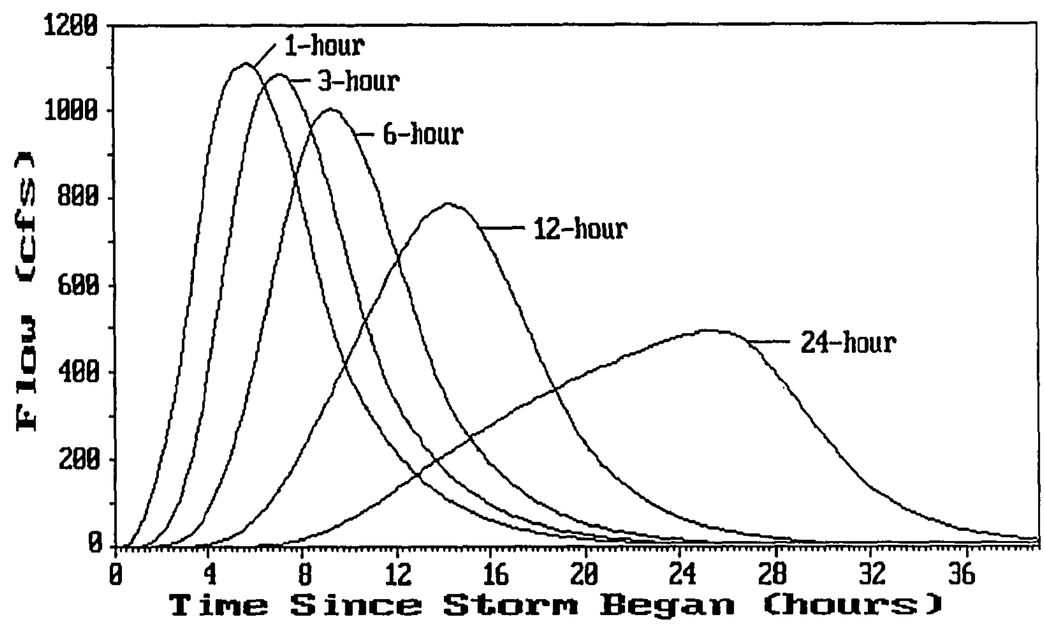

Figure 53. Hydrographs for three-inch storms with durations of 1- to 24-hours in subbasin D3

Table 18. Time in hours from initiation of basin-wide storm to arrival of peak flow at Ames for various storm durations

Antecedent Moisture Condition	Storm Duration (hours)	Time of Peak Flow Arrival
	1	12.50
	3	14.00
I	6	15.75
	12	20.50
	24	31.25
	1	12.50
	3	13.50
II	6	15.00
	12	20.00
	24	29.50
	1	12.50
	3	13.25
III	6	14.50
	12	19.50
	24	28.25

duration on arrival of the peak flow.

Changing soil moisture levels were also found to have a small impact on arrival time of flood peaks at Ames. Changing antecedent moisture conditions generally accelerated or retarded arrival of peak flow by less than an hour, however, for storm durations of 12 hours or less.

Direction of Storm Travel

As originally suggested by Lara and Heinitz (U.S. Dept. of the Interior 1976), the response of the Squaw Creek basin is somewhat dependent on the direction in which a storm travels. As seen by comparing Figures 54 and 56, a storm that travels from north to south along the main channel results in substantially higher peak flow than is caused by the same precipitation quantities distributed in a south to north pattern. Both hydrographs are for three-inch storms lasting three hours in each subbasin. In Figure 54 the rainfall was applied sequentially from south to north as illustrated in Figure 55. The north to south pattern illustrated in Figure 57 resulted in the predicted hydrograph in Figure 56.

The cause of the nearly 2000 cfs difference in peak flows shown in Figures 54 and 56 is explained by the subbasin hydrographs in these figures. A north to south storm pattern causes southern subbasin runoff to be delayed sufficiently to coincide with flow contributions from subbasins further upstream. This results in a "piling up" of hydrographs that accentuates the basin runoff response. By comparison, south to north storm patterns cause component hydrographs to move out of phase so that their superimposed flow is reduced.

Response to moving storms is also affected by storm velocity. As shown by the rainfall patterns illustrated in Figures 55 and 57, storm movement was simulated by inserting a half-hour lag between rainfall initiation in adjoining groups of subbasins. This simulates a storm that sweeps the length of the main channel in roughly two to three hours--about the same storm velocity as experienced in June

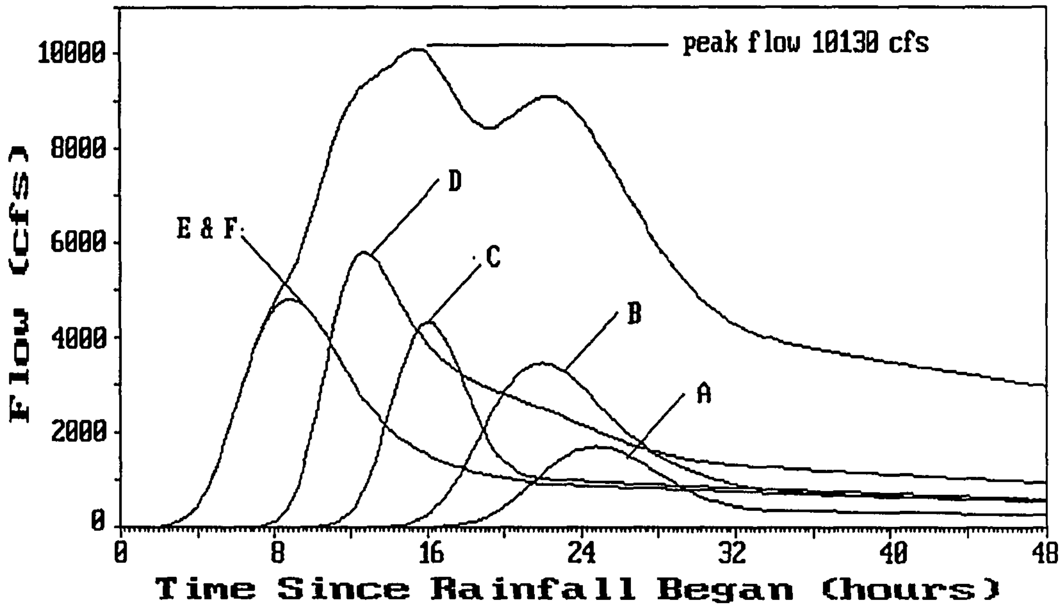

Figure 54. Subbasin and total basin hydrographs for 3-inch, 3-hour storm traveling from south to north

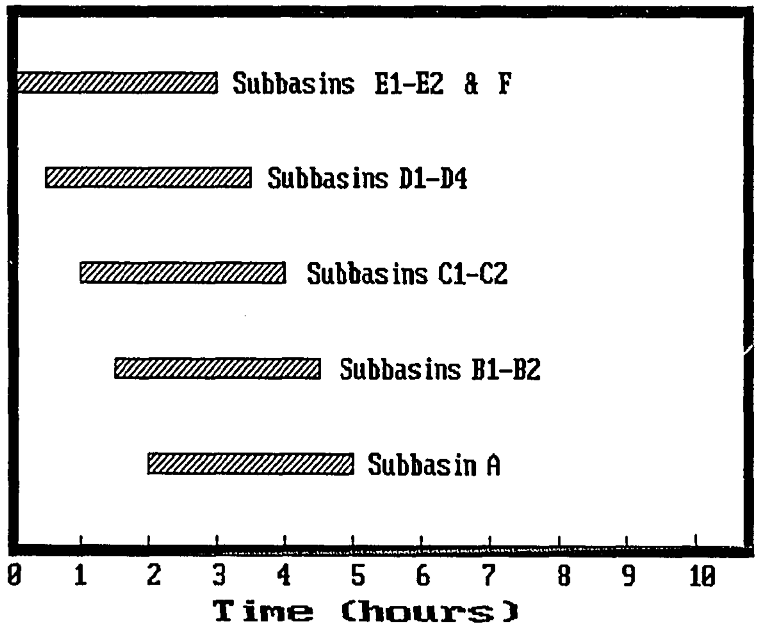

Figure 55. South-to-north rainfall pattern for hydrographs in Figure 54

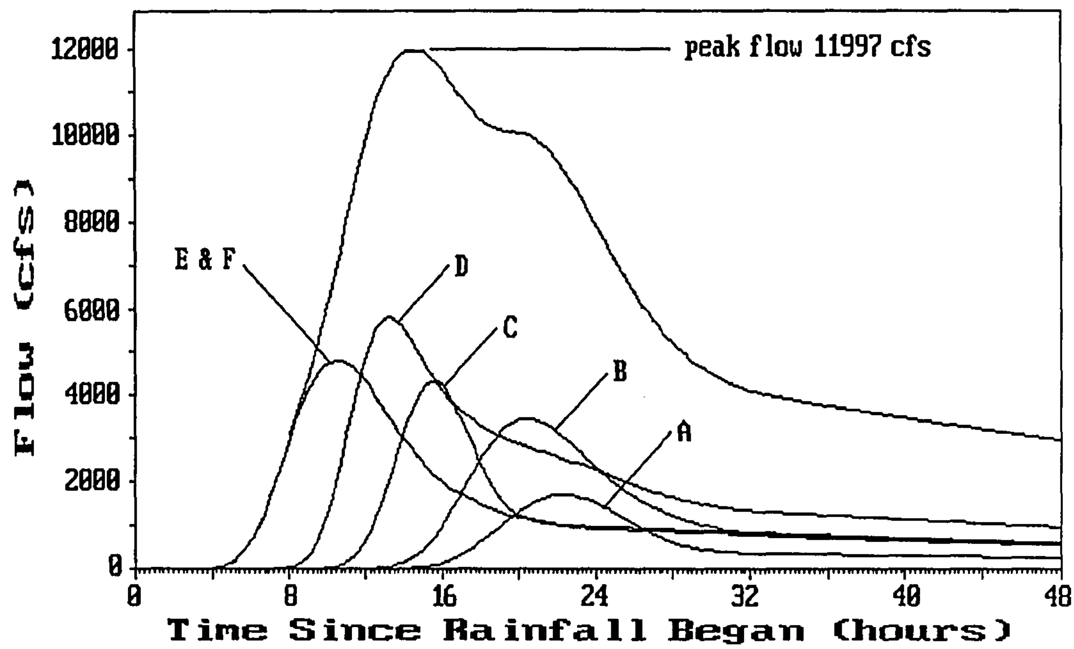

Figure 56. Subbasin and total basin hydrographs for 3-inch, 3-hour storm traveling from north to south

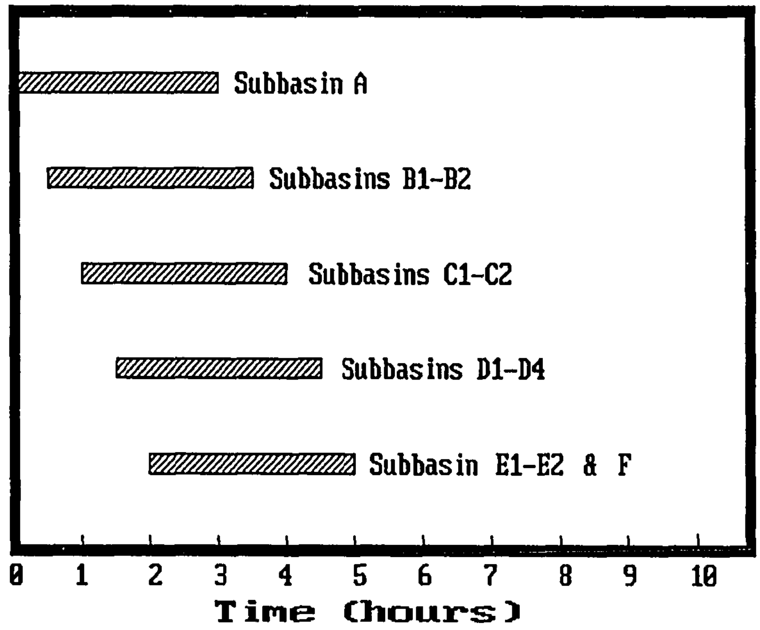

Figure 57. North-to-south rainfall pattern for hydrographs in Figure 56

of 1975 (Event 2).

For a more slowly moving storm--with a one-hour time lag between rainfall initiation in adjacent subbasins--the difference between predicted peaks for storms moving in opposite directions is nearly 3600 cubic feet per second, as shown in Figure 58.

As in earlier experiments with the duration of static storms, changes in storm length have little effect on the peak flow generated by moving storms. Figure 59 shows predicted hydrographs for one-hour duration storms moving at the same velocity as the three-hour storms in Figures 54 and 56. Peak flows predicted for one-hour and three-hour storms traveling in the same direction are nearly the same.

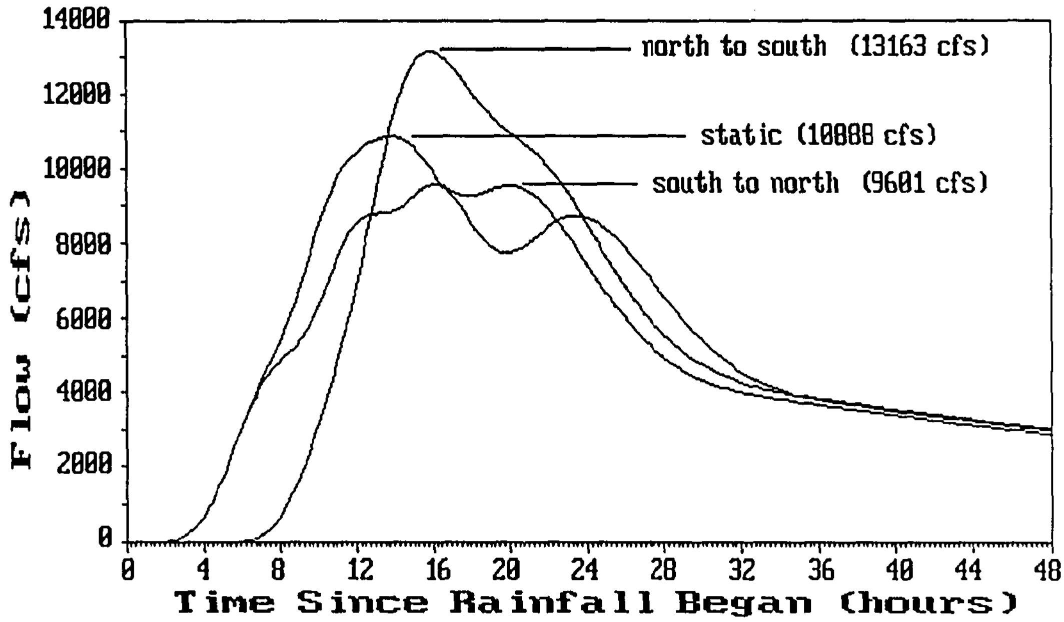

Figure 58. Hydrograph for 3-inch, 3-hour traveling storm with one-hour lag between subbasins

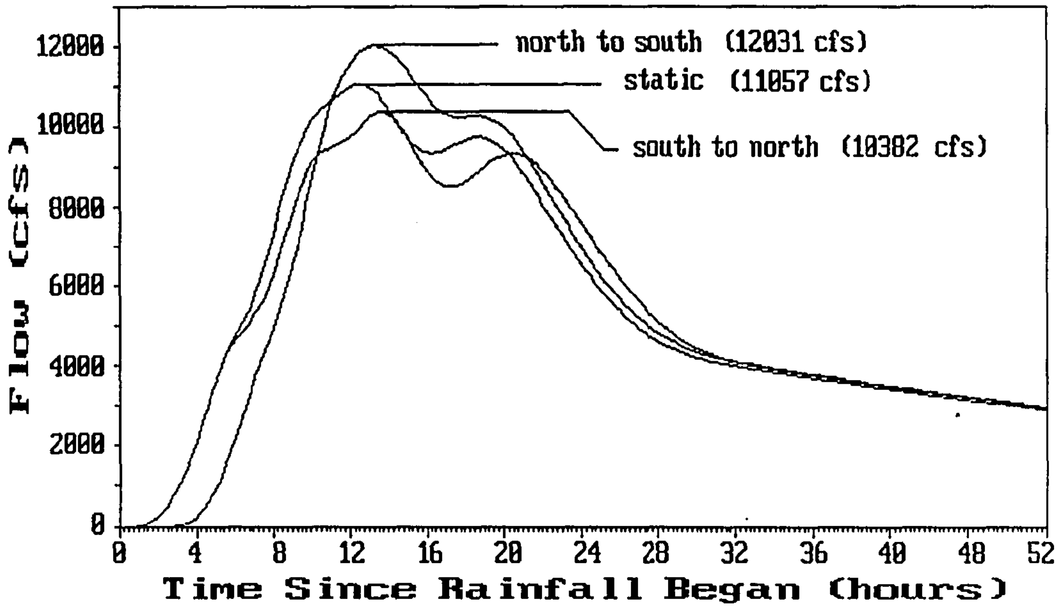

Figure 59. Hydrograph for 3-inch, 1-hour traveling storm with one-half hour lag between subbasins

PLANNING AND DEVELOPMENT OF OPERATIONAL FORECASTING METHOD

Basin response modeling shows that quantity of rainfall is only one of several hydrometeorological variables that can significantly affect flooding on Squaw Creek. In addition, antecedent moisture conditions, spatial variability of rainfall, direction and velocity of storm movement, and--to some extent--storm duration, all have noticeable impacts on the timing and amplitude of the flood crest at Ames.

Manual Forecasting Methods

Lumped Models

For uniform basinwide storms, a simple graphical or tabular flood crest prediction method, similar to the headwater tables discussed earlier (see Review of Literature), can accommodate many of the key hydrometeorological factors listed above. Figure 60, for example, was derived from numerous applications of the HEC-1 model assuming various antecedent moisture and precipitation levels. A storm duration of three hours was used to generate these peak flow predictions. As shown previously, storm durations of one to three hours do not significantly change peak forecasts for a given amount of rainfall.

Since streamflow occurring at the time the storm begins has significant impact on the resulting peak flow, baseflow prior to the crest has been eliminated from the predictions in Figure 60. To make a peak flow prediction for any isolated storm event, one simply adds the peak flow prediction read from Figure 60, to the approximate streamflow occurring at the time the storm began. For example, given slowly declining baseflow of 500 cubic feet per second and a three-inch basinwide storm occurring on a dry basin (AMC I conditions), the HEC-1 model for Squaw Creek predicts peak flow of 4050 cfs. Using the AMC I line in Figure 60, a three-inch rainfall is predicted to yield peak flow of approximately 3500 cfs. Adding roughly 500 cfs of baseflow, a peak of 4000 cfs is predicted which agrees well the HEC-1 model. Naturally, this procedure works only for isolated events preceded by slowly declining streamflow. If the event of interest occurs on steeply rising or falling portions of a hydrograph from a previous storm, the assumption of nearly constant "baseflow" is in error.

This manual peak flow estimating procedure, or ones similar to it, can be extended to cover other storm characteristics that occur on a basinwide scale. A series of graphs similar to Figure 60, for example, could be prepared to account for storms traveling along a north-south path at various constant velocities. Another series might be prepared for storm durations greater than three hours.

Manual procedures do not work well, however, for non-uniform storm or basin conditions. As illustrated by precipitation data for the events used to calibrate and

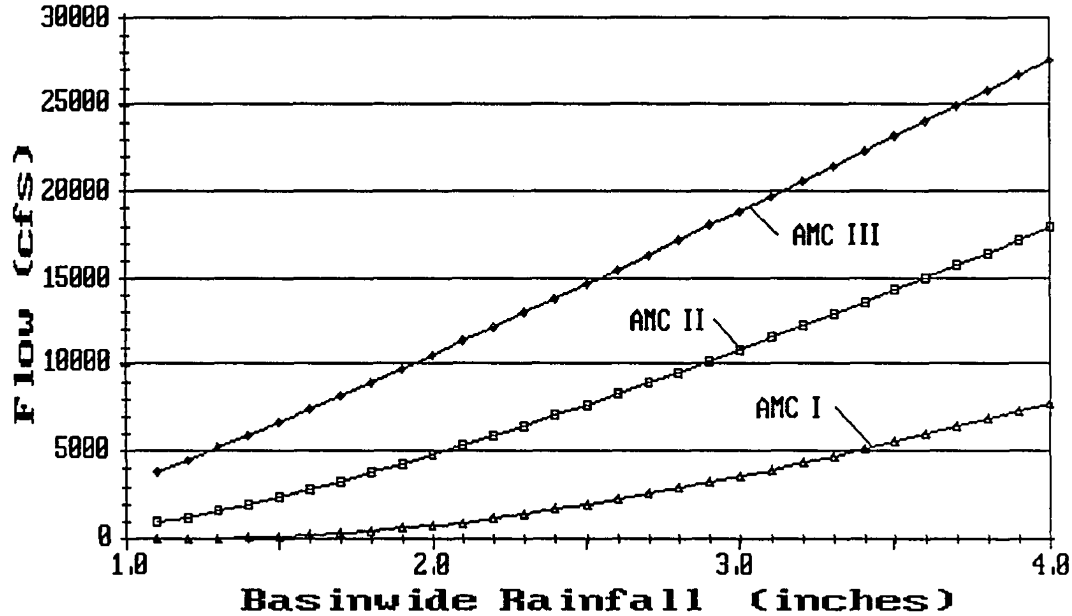

Figure 60. Peak flow predictions for various rainfall and antecedent moisture conditions

verify the HEC-1 model, heavy rainfall in the Squaw basin is often localized, as are antecedent moisture conditions. And, as demonstrated by modeling of localized rainfall, flooding in Ames can be caused by heavy rainfall over two or more subbasins--particularly if AMC III conditions are in effect.

Unfortunately, there is no simple procedure for adjusting the predictions of a simple lumped parameter model, like that represented by Figure 60, to account for spatially or temporally varied inputs.

Simple rainfall averaging, for example, produces poor results. Consider, for example, a two-inch storm localized over the center of the basin (two inches of rain on subbasins C1,C2,D1,D2,D3,D4 and one inch over the remainder of the basin). For AMC II conditions, the HEC-1 model predicts peak flow of 4012 cfs. The areally weighted basinwide average rainfall for this storm is 1.46 inches which, according to Figure 60, would cause peak flow of only about 2200 cfs.

The discrepancy is due to the fact that over 0.5 inches of runoff comes from the subbasins receiving 2.0 inches of rainfall, while less than 0.1 inch is derived from the remaining subbasins. Since peak flow is a linear function of runoff, flow contributed by the central subbasins dominates the hydrograph for the total basin. Simple averaging of rainfall over the total basin neglects the rapid increase in runoff and peak flow that occurs with each added rainfall increment.

Distributed Models

Spatial and temporal variability in rainfall and antecedent moisture conditions are generally dealt with by breaking a basin into subregions that are more likely to display uniform hydrometeorological characteristics. Separate runoff hydrographs may then be predicted for each subbasin and summed to obtain a composite flood prediction for the total basin.

While conceptually simple, this can be tedious and time consuming if performed manually. Experiments with a manual approach revealed several practical difficulties that limit its potential for use by non-technically trained persons that are unaccustomed to involved sequential calculations.

First of all, subbasin peak flow estimates derived from graphs like Figure 60 are not sufficient. Since peak flow contributions from each subbasin occur at different times, complete hydrographs must be superimposed to determine the amplitude and timing of the composite peak.

Triangular approximations to the component hydrographs were experimented with in order to minimize the number of manual calculations. For runoff sequences in which component peaks occur simultaneously, triangular hydrographs worked reasonably well. But, as the lag between component peaks increased, triangular hydrographs gave only fair estimates of peak flow and time of peak. Inability to realistically simulate the rising and falling limbs of the subbasin hydrographs seemed to be the major cause of this problem.

Another complication associated with manual development of composite flood hydrographs is that component hydrographs from each subbasin must be appropriately positioned along a common time line according to their estimated times of arrival, so that their summation will yield an accurate composite hydrograph for the total basin.

Finally, numerous summations must be made either graphically (using dividers) or by interpreting component hydrograph values at regular intervals and summing them mathematically. Although these calculations are not difficult, they are quite time consuming, particularly if the basin is divided into four or five subbasins.

Automated Forecasting Methods

In light of the operational difficulties associated with manual forecasting methods, it appears that they are not well suited for use by non-professional forecasters in basins, like Squaw Creek, that are responsive to several hydrometeorological variables. Even in simple cases involving uniform conditions throughout the basin, a forecaster must be able to correctly identify the existing combination of five variables (antecedent moisture, rainfall amount, storm duration, storm travel speed, and direction of storm travel) at the time of the storm. Then, assuming that tabular or graphical forecasting tools are developed to account for the effects of these variables, the forecaster must select and apply the appropriate table or graph that best matches the existing combination of variables.

In more complex cases involving spatially or temporally varied conditions, the forecaster must have sufficient experience to adjust predictions that are based on assumption of uniform basinwide conditions, or break the basin into subbasins that exhibit more uniform characteristics and generate a composite basin hydrograph from predicted subbasin contributions.

Recognizing that the threat of flooding is highly seasonal, often with six months or more between flood watch periods, it seems doubtful that personnel assigned to forecasting duty in Ames will attain the experience needed to competently handle complex basin conditions using manual methods. With this in mind, it was decided to explore development of a microcomputer-based flood simulator that can assist in organizing data, developing forecasts, and interpreting the forecast in light of local conditions.

Since microcomputers have become a common tool in many business and engineering applications, their accessibility for occasional flood prediction is not expected to be a problem. Furthermore, a microcomputer-based prediction tool offers several important benefits that manual procedures lack. These include:

1. Forecaster training and experience requirements are reduced since complex or tedious analysis procedures are automated;

2. Consistent forecasting procedures are applied regardless of personnel changes;

3. The effects of spatially and temporally varied inputs and complex combinations of several basin parameters are more easily cataloged and presented to the forecaster;

4. Automated forecasts can present a complete flood hydrograph, making it possible to predict flood duration as well as the flood crest; and

5. When the values of input parameters are uncertain, automated procedures allow forecasters to quickly

make predictions for a variety of likely storm conditions thereby bracketing the range of streamflows that may occur.

When considering development of a computerized flood simulator, use of packaged generalized basin models, like HEC-1, was first considered. It was decided, however, that a simplified model designed specifically for Ames and the Squaw Creek basin was justified for several reasons:

1. Generalized hydrologic modeling packages often require large amounts of computer memory, whereas a model designed specifically for a particular basin or purpose can be more easily used on an average sized microcomputer found in most offices; (Note: a microcomputer version of HEC-1 requiring 512 kilobytes of random access memory was released by the Army Corps of Engineers in 1985.)

2. Complex input formats required by generalized hydrologic models are difficult for occasional or non-technically trained forecasters to use; and

3. Output from generic hydrologic models can be difficult to interpret, whereas a program that is custom designed for a particular community can be formatted to present output in a manner that is easily related to the local situation.

Development of Automated Flood Simulator

Recognizing the difficulties facing a non-professional forecaster who must make occasional flood predictions with little or no technical assistance, a computerized flood simulator was developed with the following objectives:

1. The program must be capable of predicting streamflow resulting from non-uniform rainfall and variable basin soil moisture conditions;

2. The program must run on a microcomputer;

3. The data input process should be menu-driven for simple operation;

4. Program output should clearly indicate the value of the predicted peak flow, its time of arrival, and critical elevations within damage centers that are expected to be flooded; and

5. The program should be capable of handling flood prediction for consecutive storms.

Preliminary Assumptions

As development of the forecasting procedures was begun, it became necessary to make some assumptions about the availability of input data.

Instantaneous streamflow data is currently available, via telephone line, from the Squaw Creek gaging station in Ames. It is assumed that this data, or its equivalent will continue in the future.

It was further assumed that a sufficient number of rainfall reporting stations would be set up in the basin to provide accurate and timely information on heavy rainfall events and moderate rainfall during critical periods of unusually high streamflow. Since there is only one National Weather Service precipitation station in the Squaw basin, it was assumed that the city of Ames would set up several more.

Since fully automated precipitation measuring stations are expensive to purchase and maintain, it is likely that a network of observers will be set up to supply rainfall data during critical flood watch periods. From an operational standpoint, rainfall reports during flood watch periods must be more frequent than every 12 hours as this is the approximate response time for a basinwide storm. On the other hand, it may be difficult to get observers to immediately report storm totals for events occurring during the early morning hours.

For preliminary planning purposes, it was assumed that rainfall reports would be supplied at least every six hours during flood watch periods. Though not ideal, it is believed that this would be adequate in most cases. In instances where major storms occur during the early part of a six-hour reporting period, a four- or five-hour delay in reporting would still afford a seven- to eight-hour warning period prior to the flood crest. In cases of heavy localized rain over subbasins near Ames, a six-hour reporting delay would reduce the warning period greatly since these basins can crest at Ames in six to eight hours.

It is also presumed that rainfall observers will keep daily rainfall records so that they can report precipitation totals for the five-day period preceding major storms. This will facilitate use of the SCS Method for approximating soil moisture conditions and estimating storm runoff.

Program Structure and Operation

The general structure of the operational flood simulator is illustrated in Figure 61. The program, which is written in BASIC (programming language), is documented in the

Appendix of this dissertation. Like the HEC-1 model used for the basin study, the operational model makes use of SCS methods for runoff estimation. Runoff, peak flow, time of occurrence of the peak at Ames, and a projected hydrograph at Ames, are calculated sequentially for each subbasin. Hydrographs for each subbasin are positioned along a common time line based on the projected arrival time for their respective peaks. After the subbasin hydrographs are correctly positioned on the time line their ordinates are summed to obtain a composite hydrograph for the total basin. As many as five separate sets of storm data may be entered to simulate extended rainfall or separate storms that occur consecutively.

Although the general structure of the flood simulator may look similar to other basin models, it should be emphasized that it is not, nor is it intended to be, a detailed hydrologic model. It makes use of several computational shortcuts, based on observations of output from the more detailed HEC-1 model, to produce reasonable estimates of flood hydrographs at Ames. As such, the flood simulator is a relatively simple forecasting tool that bridges the gap between complicated basin models and manual forecasting methods. Although it is not as versatile as HEC-1 or other hydrologic modeling software, it is useful over a broader range of conditions than manual methods, it

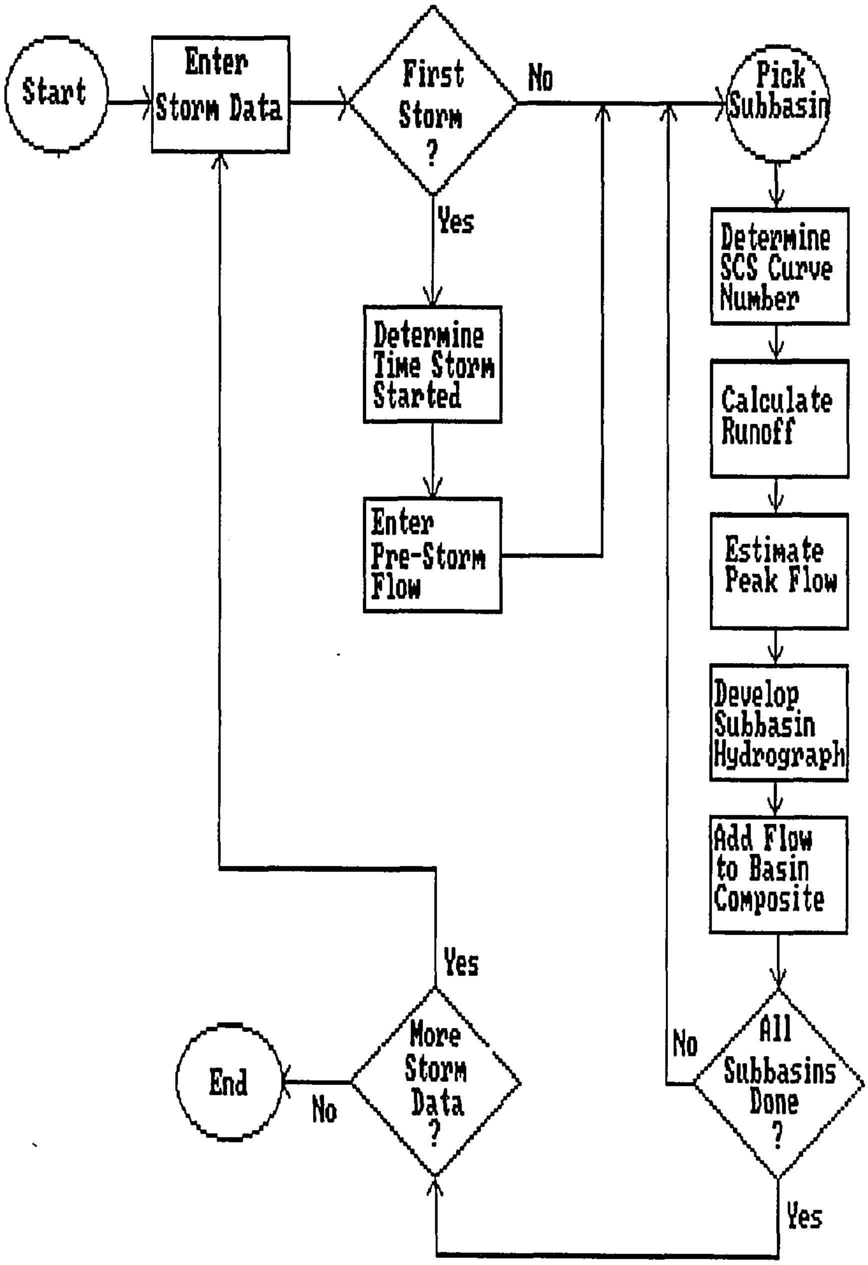

Figure 61. Flow diagram for micro-computer-based flood simulator developed for the city of Ames

reduces the potential for computational or judgmental errors by non-professional forecasters, and it presents predictions in a meaningful format for the non-professional forecaster.

Input Data   Storm data requirements include date and time of storm initiation, estimated storm duration, quantity of precipitation, and five-day antecedent rainfall. These data must be entered for each subbasin. At the present time it is uncertain how many rainfall reporting stations the city of Ames will set up throughout the basin. Once the structure of the rain gage network is established, a Theissen Net subroutine will be added to the flood simulator to calculate storm input values for each subbasin.

Data characterizing the streamflow prior to the storm must also be input to the model. These data can be obtained by telephone from the stream gaging station at Ames. The model presumes that a storm is preceded by slowly falling streamflow. Given two streamflow measurements several hours apart (prior to the storm), the program calculates a recession constant (K) using equation 13 and projects the pre-storm flow into the future using this recession constant.

Peak Flow Estimation   In accordance with unit hydrograph theory, the ordinates of a basin hydrograph resulting from rainfall of a specified duration are proportional to the quantity of runoff generated. So, for any particular subbasin, there is a linear relationship between peak flow and the amount of runoff generated during a storm.

A similar relationship exists for peak values exhibited by subbasin hydrographs that have been routed through the Squaw Creek channel to Ames using the Muskingham Method. Figure 62 was derived from output generated by the HEC-1 basin model. Various quantities of rainfall were applied (as three-hour duration storms) to individual subbasins to obtain outflow hydrographs. These, in turn, were separately routed to the Squaw Creek gaging station in Ames. When their peak values are plotted against surface runoff, the relationship between these parameters is nearly linear.

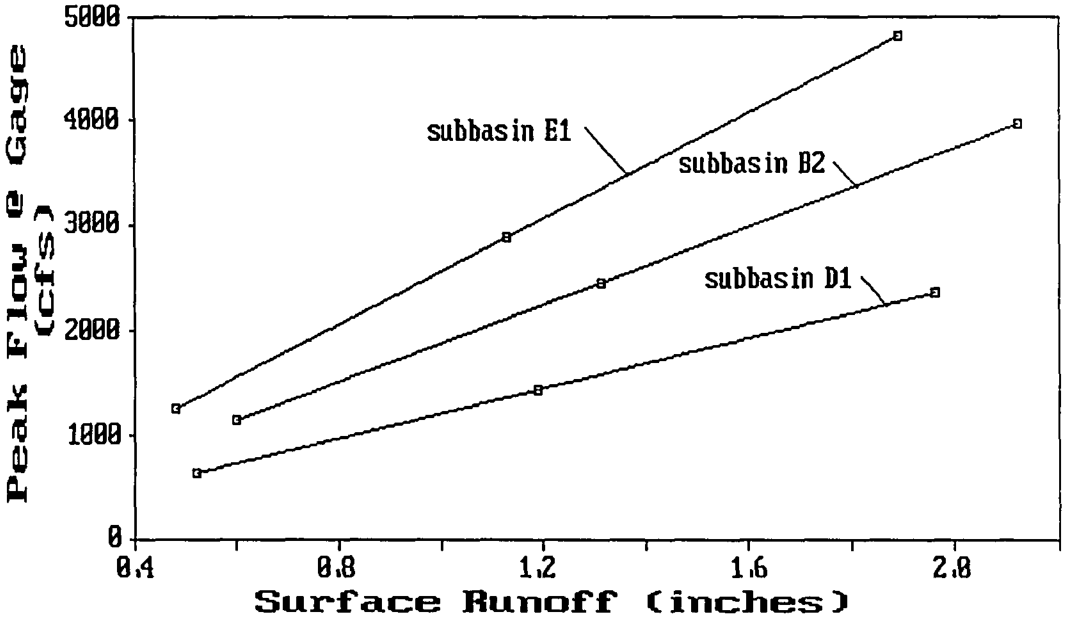

Figure 62. Peak flow at Ames versus surface runoff for three-hour storms in selected subbasins

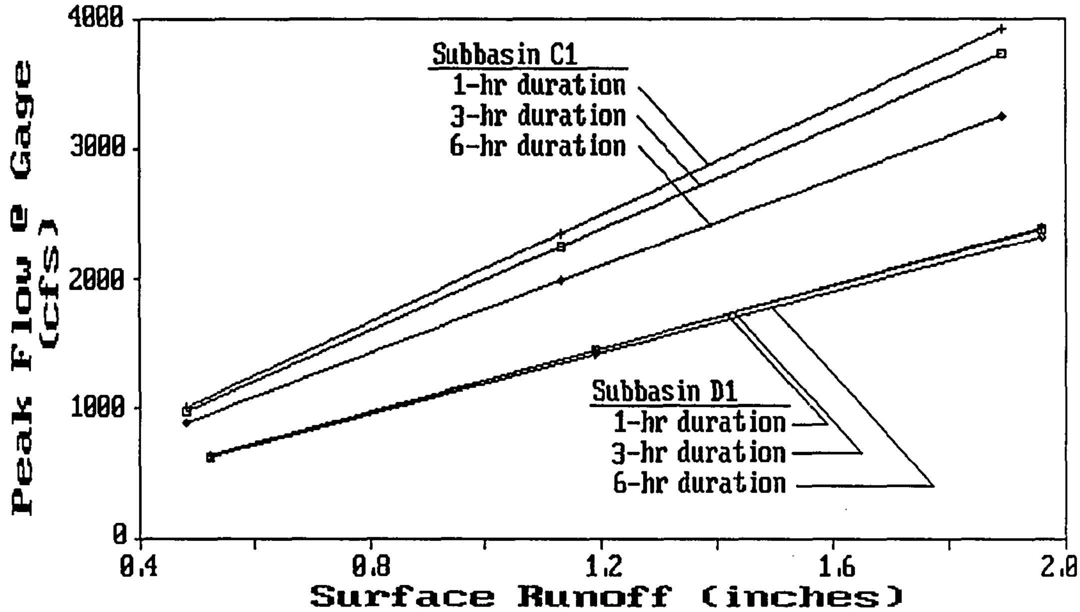

Figure 63. Predicted peak flow versus surface runoff for various storm durations

When storm duration is changed, the relationship between surface runoff and peak flow at Ames changes too, as illustrated in Figure 63. As storm duration increases, predicted peak flow decreases. The difference in predicted subbasin peak flow for one- and three-hour storms, however, is generally five percent or less. As storm duration increases to six hours, subbasins like C1 with relatively short lag times (3.1 hour estimated lag time), show a 10 to 20 percent decline in peak flow. Subbasin D1, with a lag time of 9.5 hours, shows relatively little change in peak flow for storm durations of six hours or less. Although basins become more responsive and hydrograph shape changes as antecedent moisture levels increase, the linear relationship between surface runoff and peak flow is maintained. The results of applying two-, three- and four-inch storms to subbasins C1 and D1 are shown in Figure 64 where antecedent moisture conditions were systematically varied from AMC I to AMC III. Although the predicted peak flow for any particular amount of surface runoff increases substantially with increased soil moisture, the slope of the peak flow versus runoff graph for each subbasin remains constant. The flood simulator utilizes the relatively constant linear relationship between runoff and peak flow as the basis for estimating the peak flow at Ames contributed by each subbasin.

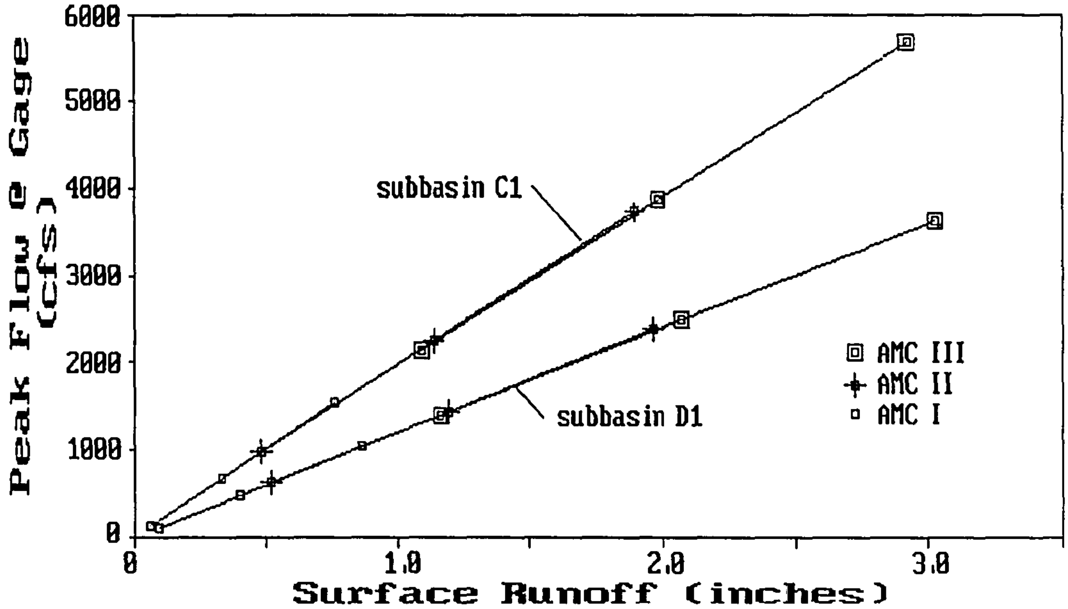

Figure 64. Peak flow versus runoff under varying AMC conditions

Since peak flow versus runoff graphs as a straight line passing through the origin, slope is the only parameter needed to identify the relationship for each subbasin. Table 19 lists the slope factor for each subbasin. These are average slopes for the peak flow versus runoff relations exhibited by hydrographs for one-hour storms. Since the slope factor declines noticeably as storm durations approach six hours, storm durations are limited to a maximum of four hours. Longer events must be broken into two or more storms, each with durations of four hours or less.

Subbasin Hydrograph Development To obtain a composite hydrograph for a large basin, detailed hydrologic

Table 19. Slope factors for the linear relationship between subbasin runoff and peak flow at Ames

Subbasin	Slope Factor (cfs/inch of runoff)
A	1317
B1	958
B2	1920
C1	2075
C2	1726
D1	1222
D2	1686
D3	832
D4	2605
E1	2625
E2	946
F	1071
G	1224

models, like HEC-1, generate large numbers of short duration synthetic hydrographs for each subbasin. These are superimposed along a common time line and summed to obtain a subbasin outflow hydrograph. Subbasin outflow graphs are routed downstream and combined with other subbasin outflow to obtain a composite flood hydrograph for the total basin.

To keep the flood simulator simple and easy to implement on a microcomputer, a shortcut to this lengthy hydrograph development procedure was sought. It was noted, during earlier attempts to develop a manual flood forecasting method, that the shape of subbasin hydrographs predicted by the HEC-1 model changes relatively little as total precipitation, storm duration, and antecedent moisture conditions change. As an example, Figure 65 shows HEC-1 hydrographs for subbasin D2 which have been routed to Ames. These are flow sequences predicted for three-hour duration rainfalls of two, three, and four inches. Naturally, there is considerable difference between the predicted peak flow from these three storms, and their hydrographs look quite different. But, when their ordinates are made dimensionless by plotting flow as a fraction of peak flow, the three graphs become nearly identical as shown in Figure 66. The greatest differences occur at the mid-range of the rising limb where q/q_0 for the three storms differs by approximately 0.05.

Changes in storm duration cause a slightly greater

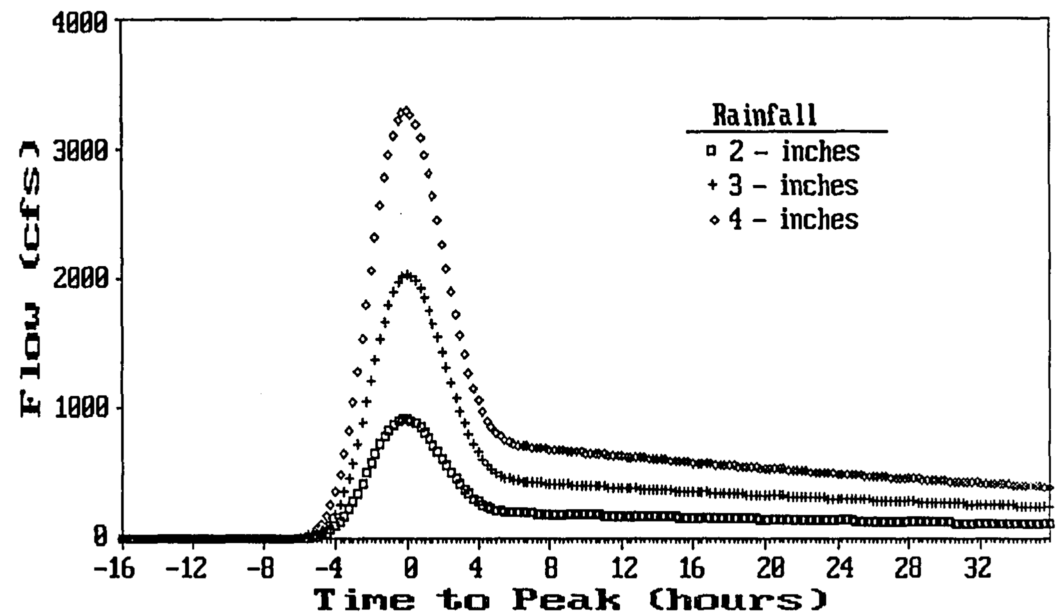

Figure 65. Streamflow hydrographs at Ames from subbasin D2 for three levels of rainfall

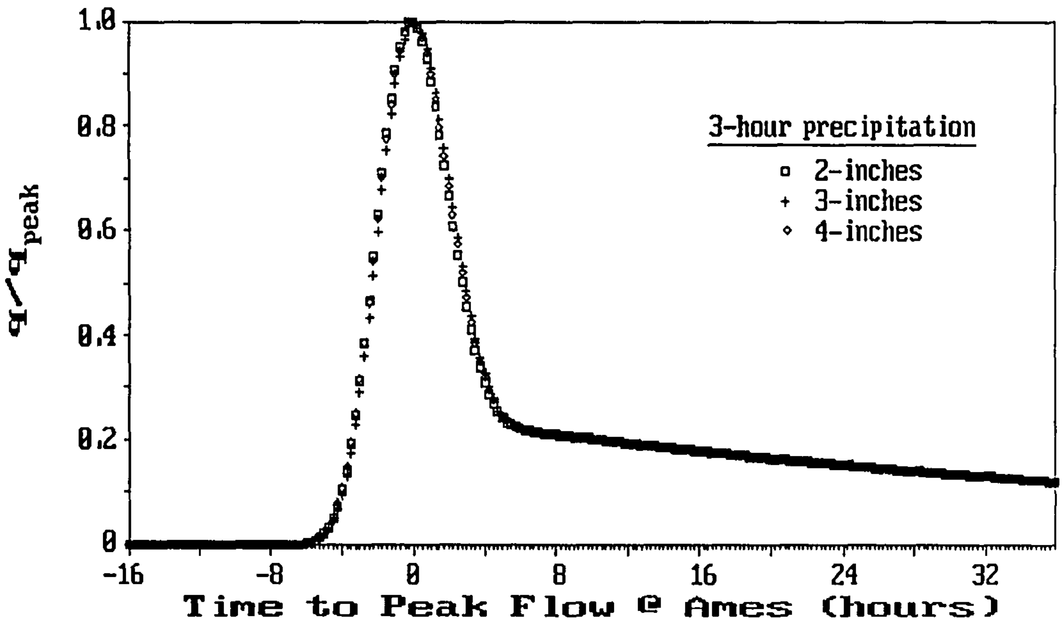

Figure 66. Dimensionless form of the hydrographs shown in Figure 65

change in the shape of the dimensionless hydrographs. Figure 67 illustrates this with routed hydrographs for subbasin D2 resulting from three-inch storms of one-, three-, and six-hour duration. Here again we see the greatest differences in the middle portion of the rising limb. At two hours prior to the peak, for example, q/q_0 ranges from 0.53 for a one-hour storm to 0.69 for a six-hour event. If storm durations are limited to a maximum of four hours, as proposed earlier to minimize variation of the peak flow versus runoff slope factor, the differences in q/q_0 are less than 0.10.

Changes in q/q_0 caused by a shift in antecedent moisture conditions are quite small. Figure 68 shows the characteristic shape for hydrographs from subbasin D2 resulting from three-inch, three-hour duration storms under AMC conditions I, II, and III. Here the maximum variation in q/q_0 across the three moisture levels is less than 0.05.

Superposition of Subbasin Hydrographs The final step in developing a flood hydrograph for the total basin is to superimpose subbasin hydrographs and sum them. To do this the component hydrographs must be correctly positioned on a common time line to reflect their different arrival times in Ames. This is accomplished using the time delays shown in Table 20. These were derived by applying one-hour duration storms to each subbasin and observing the time of peak flow in Ames as predicted by the HEC-1 model.

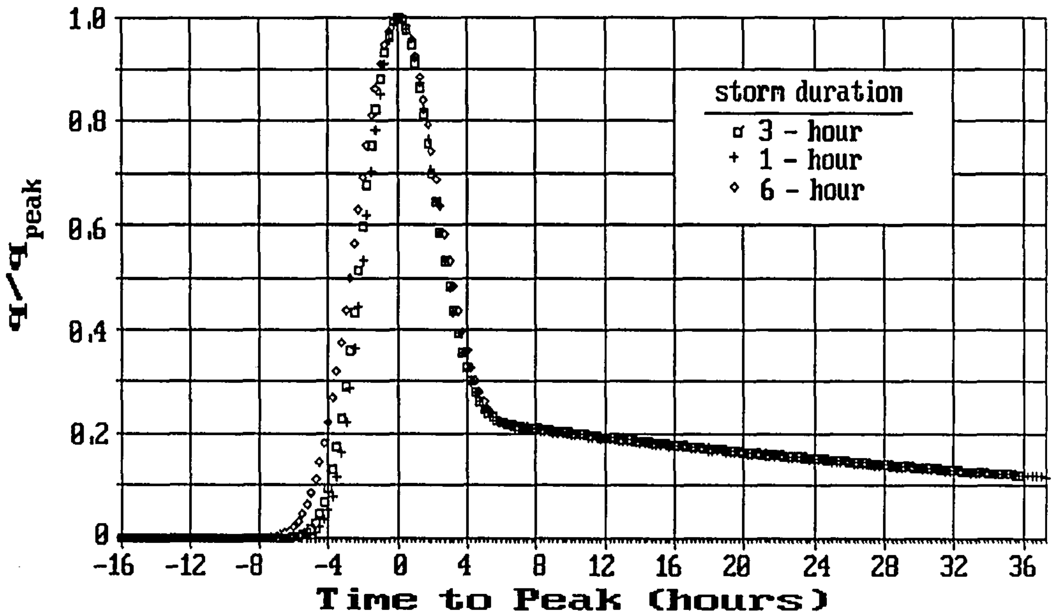

Figure 67. Dimensionless hydrographs for three-inch rainfall of one-, three-, and six-hour duration in subbasin D2

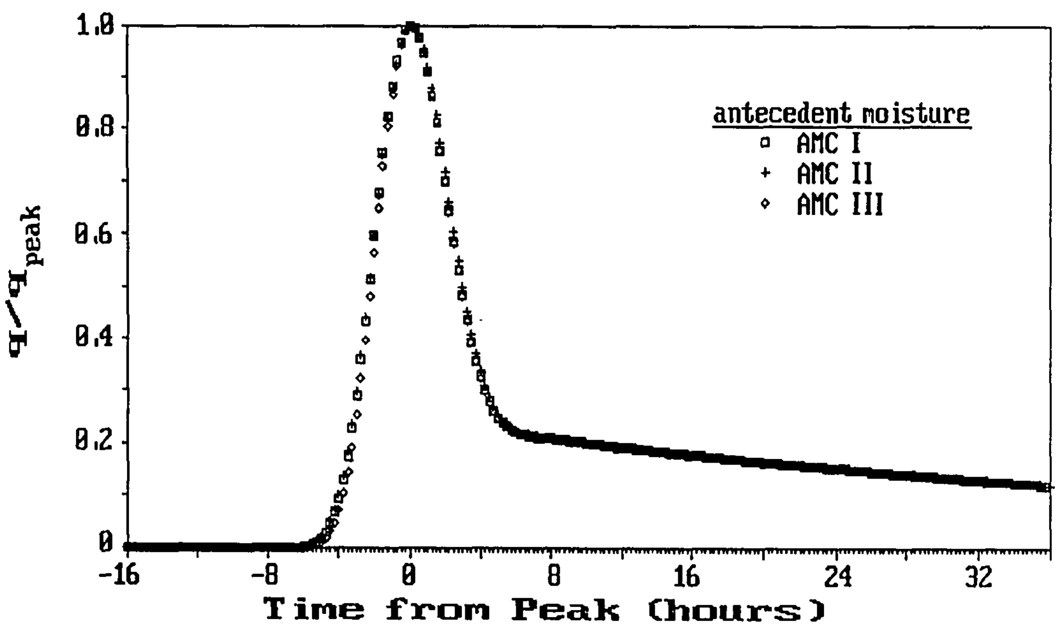

Figure 68. Dimensionless hydrographs for subbasin D2 for a three-inch, three-hour duration storm and three antecedent soil moisture levels

Table 20. Time from storm initiation to peak flow arrival at Ames

Subbasin	Time (hours)
A	20.75
B1	21.00
B2	18.00
C1	13.00
C2	13.25
D1	16.75
D2	10.75
D3	12.25
D4	9.50
E1	6.50
E2	10.00
F	7.00
G	6.50

For storm durations greater than one hour, the arrival times in Table 20 must be adjusted. Arrival times for hydrographs resulting from three-hour duration storms are delayed by approximately 1.5 hours. Similarly, peak flows caused by six-hour storms arrive about 3.75 hours later than indicated in Table 20. Peak flow time adjustments for storm durations less than six hours can be estimated using the following second degree La Grange polynomial:

$$T_a = \frac{(D-1)(D-6)}{-6}(1.50) + \frac{(D-1)(D-3)}{15}(3.75) \qquad (16)$$

Where

T_a = Adjustment (in hours) to be added to subbasin peak flow arrival times in Table 20;

D = Storm duration (in hours).

This equation forms a smooth curve through the points (1,0), (3,1.50), and (6,3.75) where the first number of each pair is storm duration and the second is the appropriate time adjustment to be added to the values in Table 20.

Performance Testing of Flood Simulator

To evaluate the accuracy of the micro-computer flood simulator, a variety of storms was input and the predictions were compared with output from the HEC-1 model developed in Phase I of the project. Four general types of storms were used to test the versatility of the simulator. These included: single storm events, multiple storm events, storms traveling along a path parallel to the main channel of Squaw Creek, and localized storms affecting a cluster of subbasins. Rainfall amount, storm duration, and soil moisture conditions were varied across the anticipated spectrum of values to test the useful operating range of the micro-computer program.

Single-event Storms

Table 21 summarizes the single event storms that were tried. In all cases the peak flow projected by the flood simulator was within five percent of the HEC-1 projection.

Table 21. Comparison of flood predictions using HEC-1 and the flood simulator

Storm Conditions			Predicted Peak Flow @ Ames			
			HEC-1		SIMULATOR	
Precip. (in.)	Duration (hours)	Baseflow (cfs)	Peak (cfs)	Time (hrs)	Peak (cfs)	Time (hrs)
AMC I						
2	1	0	797	13.00	798	12.00
3	3	0	3689	14.25	3733	13.25
4	1	500	8556	12.50	8580	11.75
4	6	0	7885	15.75	8156	15.75
5	3	0	13378	14.00	13634	13.25
AMC II						
2	6	0	4851	15.75	5032	15.75
3	3	0	11209	13.75	11448	13.25
4	1	0	18732	12.50	18920	11.75
AMC III						
2	3	0	10680	13.50	10954	13.25
3	1	0	19412	12.50	19636	11.75
3	3	500	19586	13.25	20052	13.25
4	6	0	27119	14.50	28728	15.50

The estimated time of arrival of the flood crest given by the simulator was within one hour of the HEC-1 forecast. Figure 69 shows predicted hydrographs for a four-inch one-hour basinwide storm under AMC I moisture conditions. Figure 70 is for a three-inch three-hour storm under AMC III conditions. Note that, in both cases, the flood simulator does a reasonably good job of matching the amplitude and shape of the HEC-1 prediction.

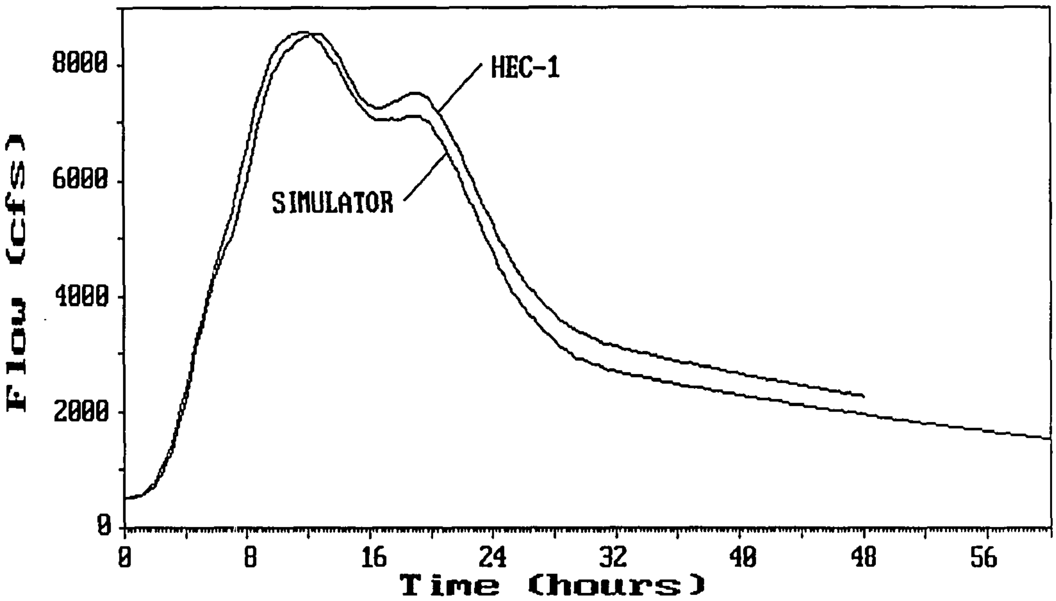

Figure 69. Predicted hydrographs for 4-inch, 1-hour basinwide storm with starting baseflow of 500 cfs and AMC-I moisture conditions

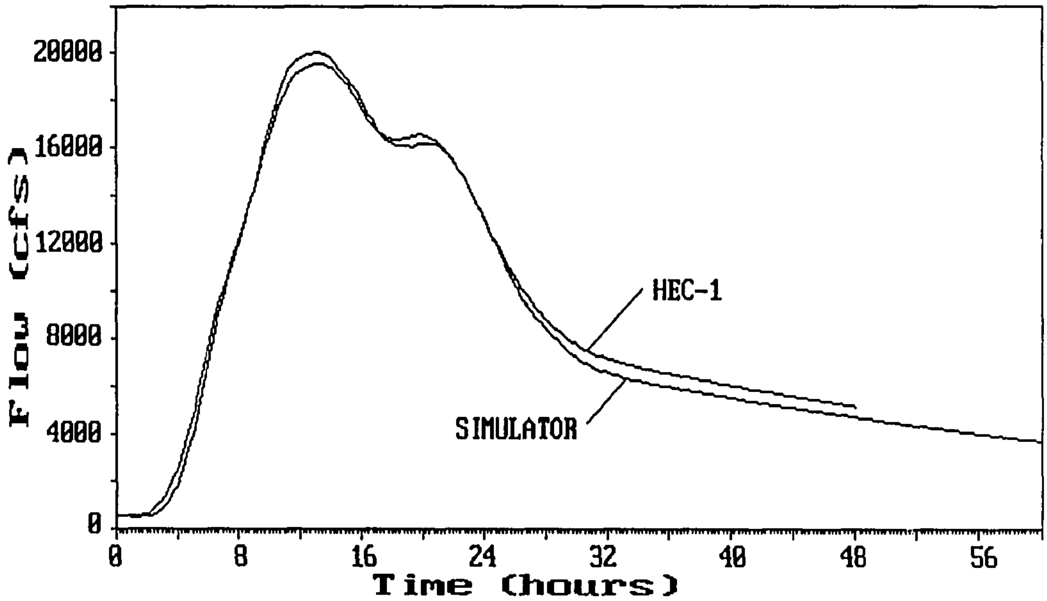

Figure 70. Predicted hydrograph for 3-inch, 3-hour storm with 500 cfs initial baseflow (AMC III)

Localized Storms

Figures 71 and 72 display flood predictions for localized heavy rainfall in northern and southern regions of the Squaw basin. In Figure 71 three inches of rainfall was distributed over subbasins A,B, and C during a three-hour time span. One inch of precipitation fell in the remainder of the basin. In Figure 72, the three-inch storm was applied to subbasins E,F, and G, and the northern subbasins received one inch. As in the case of basinwide storms the flood simulator provides good estimates of the flood crests that are within two percent of the HEC-1 results.

Traveling Storms

Two storms that move parallel with the spine of the basin were tested. Figure 73 shows the results for a three-inch two-hour duration storm that sweeps the length of the basin from north to south in approximately three hours (time from first rainfall till end of all rain in the basin). Figure 74 is for the same rainfall amount applied in a south to north pattern. The storm pattern was achieved by placing a fifteen-minute delay in storm initiation between adjoining basins. Note that both the HEC-1 model and the flood simulator predict approximately 1,100 cfs difference in peak flow between these storms that are moving in opposite directions.

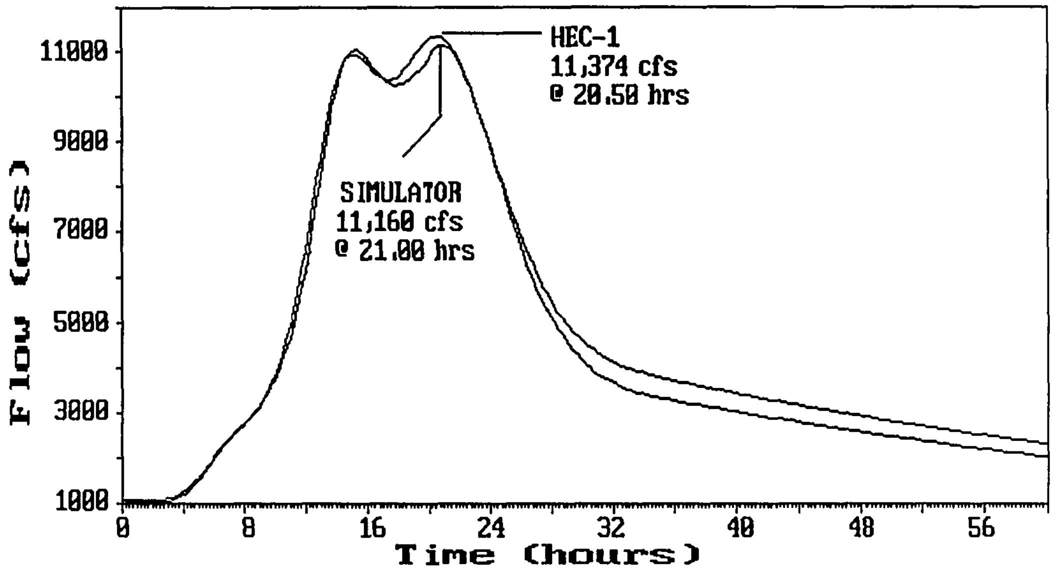

Figure 71. Flood forecast for 3-inch, 3-hour storm over subbasins A,B, and C with remainder of basin receiving 1 inch of precipitation (AMC III with initial flow of 1000 cfs)

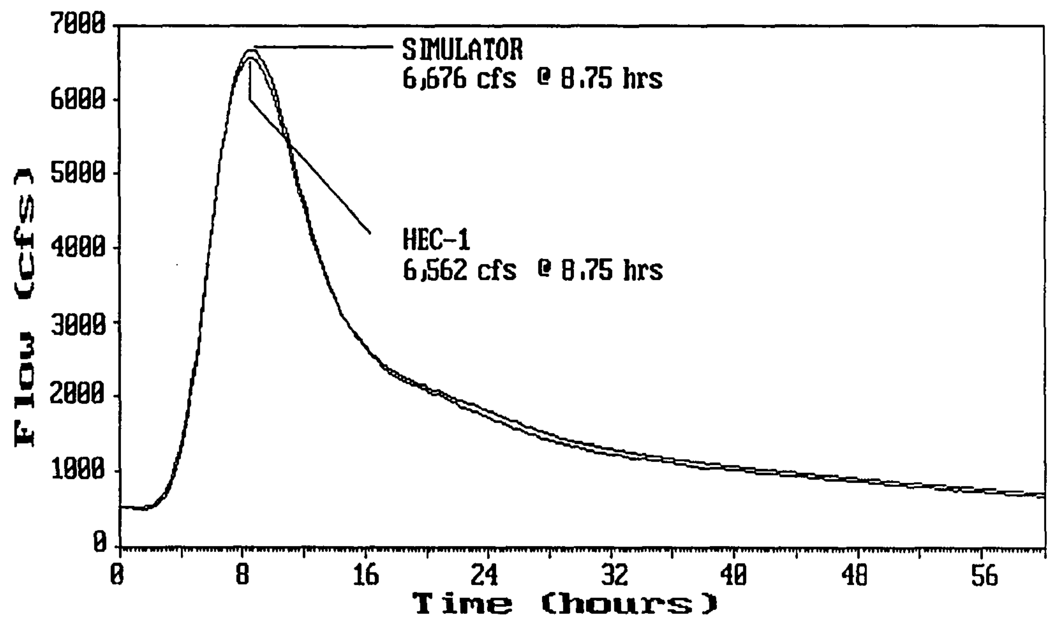

Figure 72. Predictions for 3-inch, 3-hour storm over subbasins E,F,and G, with 1 inch in remainder of basin (AMC-II with initial flow of 500 cfs)

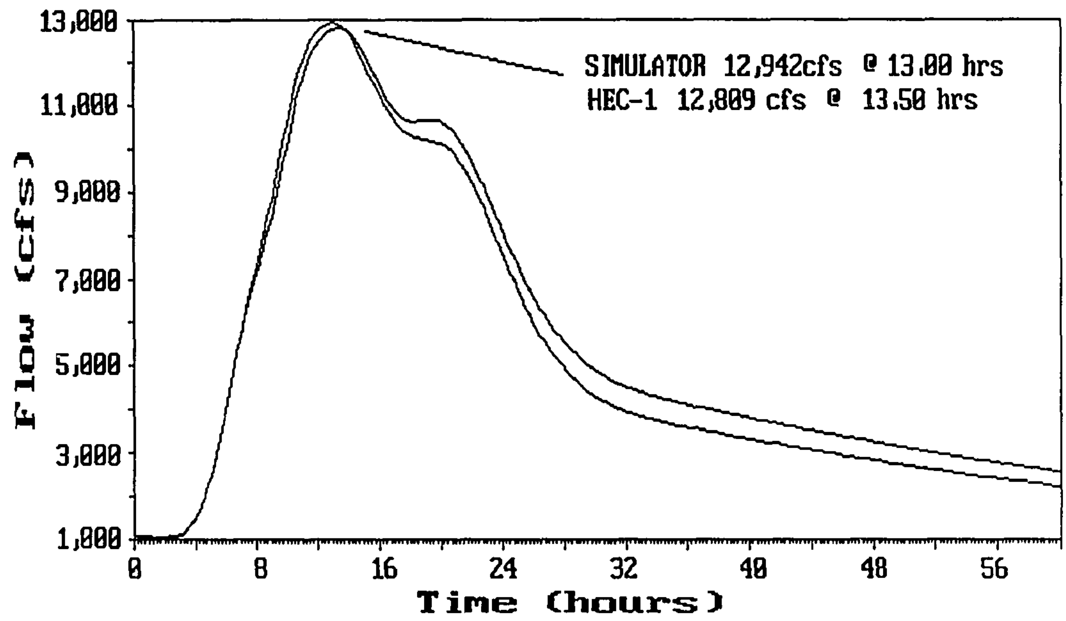

Figure 73. Predicted hydrographs for 3-inch, 2-hour basinwide storm traveling from north to south

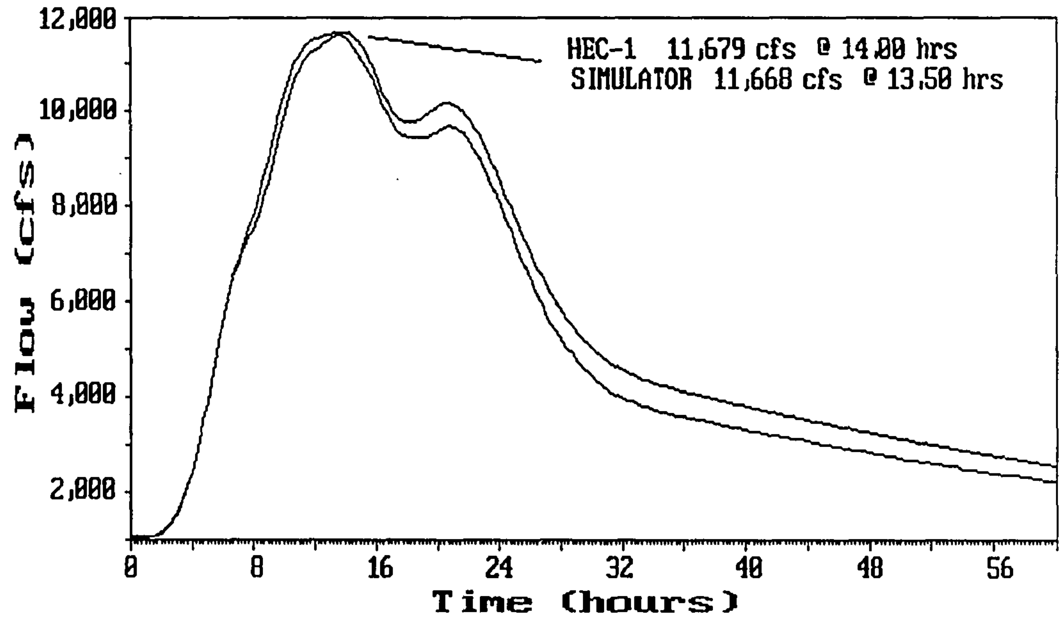

Figure 74. Predicted hydrograph for 3-inch, 2-hour rainfall basinwide storm traveling from south to north

Multiple Storms

As illustrated by the record flood of 1975, large floods are often caused by a series of storms occurring over a one- or two-day time period rather than by a single rainfall event. To handle this type of occurrence, the flood simulator was designed to permit entry of up to five separate storm events during a 48-hour time span. These may be entered during a single session at the micro-computer or the predicted hydrograph from the early storms can be saved to disk and called up at a later time so that additional storm data can be added and a new composite hydrograph generated.

Figure 75 compares the output from the flood simulator and the HEC-1 model for two storms occurring in a seven-hour

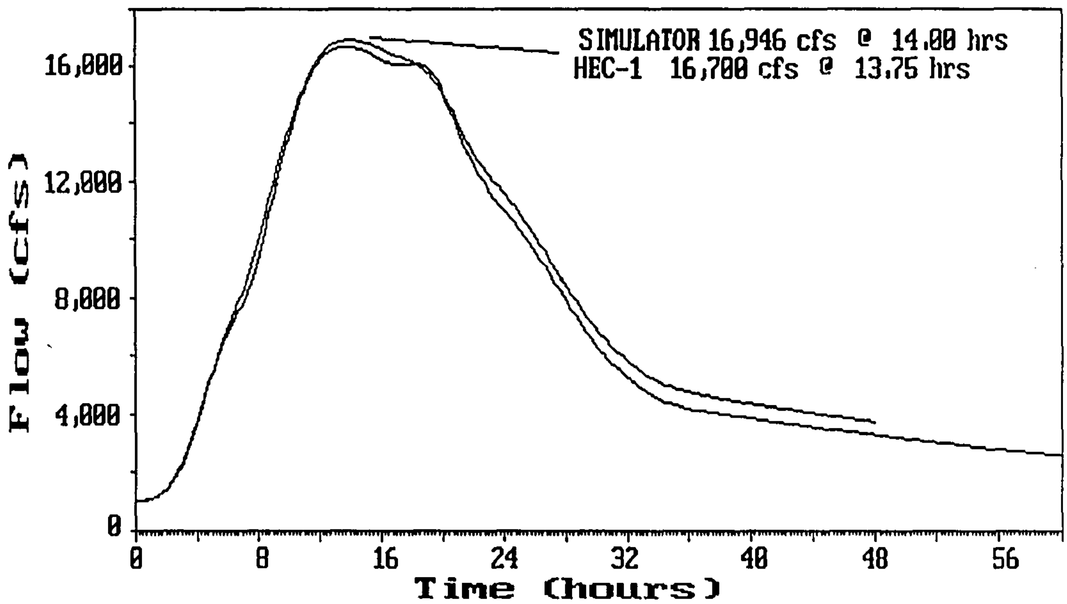

Figure 75. Predicted flow for two consecutive storms

period. The first storm is a three-inch, one-hour basinwide storm that began at midnight. AMC II conditions were assumed, and streamflow at Ames preceding the rainfall was 1000 cfs. A second one-inch one-hour duration rainfall, beginning at 0600 the same day, was superimposed on the earlier flow.

Figure 76 shows the results of a three-storm sequence. The first two events were identical to those used to generate the hydrograph in Figure 75. The third event is a two-inch, one-hour rainfall that begins 24 hours after the first event was initiated. This type of rainfall sequence is considerably more difficult to handle than the one in Figure 75 because the long time delay between the second and third storms permits many of the subbasins to start the baseflow recession phase of their respective hydrographs.

The original flood simulator was designed to begin baseflow recession (actually interflow recession, as discussed earlier) when flow drops below 20 percent of the peak flow in each subbasin. Surface runoff from subsequent storms is then added to the trailing baseflow sequence from previous events. As shown in Figure 76, the resulting hydrograph has two distinct peaks with the second of these being the larger. As indicated in the figure, the prediction of the original flood simulator over-predicts the second peak by about 12 percent.

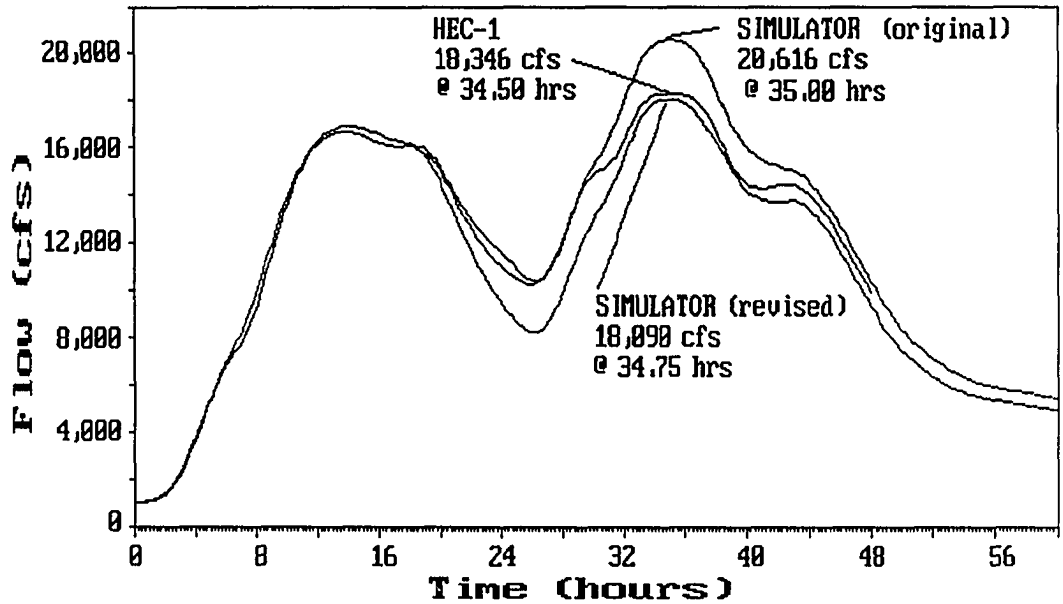

Figure 76. Predicted streamflow for a series of three storms occurring during a 24-hour period

The failure to match the HEC-1 forecast appears to be the result of how HEC-1 handles baseflow during multiple events. According to the HEC-1 users manual (U.S. Army Corps of Engineers 1985, 31), baseflow is handled in the following manner:

> "The rising limb of the streamflow hydrograph is adjusted for baseflow by adding the recessed starting flow to the computed direct runoff flows. The falling limb is determined in the same manner until the computed flow is determined to be less than QRCSN (variable specifying the percentage of the peak at which baseflow is to begin--20 percent in this case). At this point, the time at which the value of QRCSN is reached is estimated from the computed hydrograph. From this time on, the streamflow hydrograph is computed using the recession equation unless the computed flow rises above the baseflow recession. This is the case of a double

> peaked streamflow hydrograph where a rising limb of the second peak is computed by combining the starting flow recessed from the beginning of the simulation and the direct runoff."

This means that recessed baseflow occurring prior to the beginning of the first storm is added to all subsequent runoff, but that baseflow from subsequent peaks does not play a role in computing the peak flow for storms that follow them. The rationale for this procedure is not explained, but it appears that the designers of HEC-1 feel that baseflow contributions are suppressed by additional surface runoff. To test the results of this approach, the simulator was revised so that baseflow from early storm events is not accumulated and added to the rising limb of subsequent floods. Figure 76 shows the predicted second peak to be in good agreement with the HEC-1 model.

The hydrograph from the revised flood simulator does show a substantially deeper trough between the two peaks than is evident in the HEC-1 output. Although the HEC-1 program does not actually accumulate baseflow as it computes the rising limb of sequential hydrographs, it apparently does display the inter-peak baseflow in its output so that the resulting hydrograph sequence looks reasonable.

Development of Stage-Discharge Relationships at Damage Centers

The primary goal of the proposed flood prediction and warning program in Ames is to predict peak flood stage at several major damage centers along the flood plain. To accomplish this, it is necessary to relate predicted discharge rates to flood stages (water surface elevations) at the appropriate locations in the damage centers.

Identification of Major Damage Centers

The first step in relating flood stage to peak discharge, was to identify the primary damage centers within the flood plain and determine critical elevations in these areas.

To accomplish this the City Engineer was contacted about specific properties that experienced significant flood damage during the record flood of 1975. In response to this inquiry, the City Engineer provided a map of homes, businesses, and institutional structures in the flood plain that have sustained damage in past floods.

Three general damage centers could be identified from the information provided by the City Engineer. The largest damage center was a five-block long area located along both sides of South Duff Avenue. Approximately 25 business establishments are located in this area.

Further upstream, along South Maple Avenue, five homes, an apartment complex, a nursing home, and two buildings belonging to the Iowa Department of Transportation comprise the second major damage center.

The third damage center consists of two large public buildings, the Scheman Continuing Education Center, and the Hilton Coliseum, which are located in the Iowa State Center complex. Table 22 summarizes the critical elevation data for each damage center.

Development of Stage-Discharge Relationships

HEC-2, a generalized computer program developed by the U.S. Army Corps of Engineers (U.S. Army Corps of Engineers 1981), was used to calculate water surface profiles in the flood plain. All data for a calibrated and verified hydraulic model of the Squaw Creek flood plain were supplied by the Planning Division, Rock Island District Corps of Engineers. This model was originally developed by the Rock Island District Office to support a flood insurance study. The only additional input data needed to run the model were flow rates and the starting water surface elevation at the confluence of Squaw Creek and the Skunk River.

Determination of Starting Elevations HEC-2 uses the standard step method for calculating water surface profiles. This is an iterative procedure that begins with a known water surface elevation at a downstream cross-section (for sub-critical flow), and then calculates the water surface elevation at an adjacent upstream cross-section.

To initiate this computational procedure, it is necessary to know the approximate water surface at some downstream control point. In the case of Squaw Creek, the

Table 22. Critical elevations for structures within damage centers along the Squaw Creek Floodplain

Address	Type of Building or Name of Business	First Floor Entry Elevation
	South Duff Area	
906 S. Duff	Frozen Food Kitchen	887.77
816 S. Duff	Iowa Glass	887.14
814 S. Duff	Warehouse	885.65
811 S. Duff	Vacant building	885.58
806 S. Duff	A.Y. Mc Donald Co.	887.94
716 S. Duff	O'Malley & McGee's Restaurant	886.84
715 S. Duff	Fraternal Lodge	887.00
713 S. Duff	Community Building	886.88
710 S. Duff	Rent-All	886.87
710 S. Duff	Rent-All Warehouse	884.70
705 S. Duff	Layne-Western Co.	887.73
551 S. Duff	Happy Joe's Restaurant	886.64
538 S. Duff	Multi-purpose (car dealer, grocery, real estate, etc.)	
	-west door	887.07
	-north door	886.35
	-east door	887.10
	-southeast door	886.42
535 S. Duff	Smitty's	887.27
531 S. Duff	Ruttles Restaurant	889.45
520 S. Duff	Tavern	884.94
520 S. Duff	Players Lounge	884.83
508 S. Duff	APCO gasoline station	888.11
507 S. Duff	Movie Theater	886.79
505 S. Duff	Bowling Alley	889.16
118 S.E. 5th	Muffler Shop	888.00
202 S.E. 5th	Wholesale Electric Supply	885.11
214 S.E. 5th	Skating Rink	886.48
	Maple Avenue Area	
511 S. Maple	Residence	890.89
457 S. Maple	Residence	890.54
445 S. Maple	Residence	891.15
443 S. Maple	Residence	891.68
439 S. Maple	Residence	894.32
1108 S. 4th	Apartment Complex	894.33
1204 S. 4th	Nursing Home	887.43

Table 22. (continued)

Address	Type of Building or Name of Business	First Floor Entry Elevation
_[a]	Drivers License Station	
	-west entrance	897.41
	-stairwell west entrance	896.87
	-stairwell south side	896.16
_[a]	Ia. Dept. of Trans. Warehouse	893.79
Iowa State Center		
_[a]	Scheman Continuing Ed. Bldg.	
	-loading dock	892.84
	-west door	892.94
	-north door	895.32
	-south door	896.40
_[a]	Hilton Coliseum	
	-loading dock[b]	881.08
	-bottom south door	891.70
	-west door	896.43

[a]No address given with elevation data provided by City Engineers's Office.

[b]lip of drive leading to loading dock is at elevation 897.37. This elevation would have to be exceeded before the loading dock can be flooded.

downstream control point is its confluence with the Skunk River. As such, the concurrent flow and water surface elevation on the Skunk could conceivably effect the water surface profile along Squaw Creek. If flow in the Skunk is high, the starting water surface elevation on Squaw Creek will be higher than if low flow is occurring in the Skunk Channel.

A series of water surface profiles provided by the Rock Island District Corps of Engineers shows that water surface elevations at the confluence of the Skunk and Squaw are expected to vary from 880 to 884 feet (mean sea level). These elevations were computed assuming simultaneous five-year and 500-year floods on both streams.

To test the sensitivity of Squaw Creek water surface profiles to changes in the starting elevation, a series of computer runs was made with values ranging from 880 to 884 feet. This analysis showed that computed water surface at South Duff Avenue fluctuates by less than one-half foot when starting elevations vary between 880 and 884. At cross-sections further upstream, the differences in water surface elevation were only a few hundredths of a foot.

Based on these results, it was concluded that fluctuating flow in the Skunk River has minimal effect on water surface elevations at damage centers along Squaw Creek. For practical purposes, a single starting water surface elevation may be assumed without significant error. As a result, all subsequent water surface computations were started at elevation 883.

Stage-Discharge Tables at Damage Centers To determine the stage associated with various discharge rates on the Squaw, a series of HEC-2 runs was made with flow rates varying from 2000 cfs to 14,000 cfs. According to flood frequency calculations by Lara and Heinitz (U.S. Dept. of the Interior 1976) a 2000 cfs flood has a return period of less than 2 years, and 14,000 cfs exceeds the 500-year event. Table 23 summarizes the calculated stage discharge relationships at each of the damage centers.

Table 23. Stage-Discharge Table for Damage Centers

	Stage (feet above mean sea level) at Major Damage Centers		
Discharge Rate (cfs)	South Duff	South Maple	I.S.U. Center
2000	883.24	885.73	888.45
4000	883.88	888.43	891.60
6000	884.72	890.23	893.32
8000	885.61	891.66	894.62
10000	886.47	893.07	895.80
12000	887.27	894.44	897.00
14000	887.99	895.81	898.36

RECOMMENDATIONS FOR PROGRAM IMPLEMENTATION AND FUTURE WORK

According to some literature on local flood prediction and warning programs, many communities and community leaders fail to recognize the safety hazards and property damage potential of local flooding unless damaging floods occur frequently. As a result, successful implementation of a flood warning program in a city like Ames--that is frequently threatened by flooding, but only occasionally damaged by it--is likely to require more thorough planning and stronger leadership than is needed in frequently flooded communities.

Successful implementation of a flood warning program in Ames will require careful attention to inter-agency coordination, data collection, well planned operating procedures, and periodic refresher training for personnel.

Inter-agency Coordination

Any flood warning program implemented by Ames should be coordinated with several local, state, and regional agencies. Both the National Weather Service in Des Moines and the North Central River Forecasting Center in Minneapolis should be consulted as a community flood warning program is developed. These agencies offer expertise in both meteorology and hydrology, making them valuable resources for program planning and implementation.

In addition, the Weather Service Office in Des Moines can provide advance warning of threatening weather conditions that could result in flooding. This information could prove valuable as criteria for triggering a flood watch by city personnel.

The Story County Office of Disaster Services can also play a vital role in planning emergency flood protection measures in the event that local flooding is predicted.

Local electronic and print media should also be asked to help implement the flood warning program. Local radio and television stations can assist in warning dissemination when a flood warning is issued. And, both types of media can help to educate the general public regarding the nature of the flood hazard in Ames, operation of the flood warning program, and appropriate emergency response measures for property owners in the flood plain.

As in the past, local law enforcement officials must continue their role as a key communication link with floodplain inhabitants.

Data Collection

Accurate and timely rainfall and streamflow data are essential for good flood forecasting. Furthermore, as coordinated storm data for the Squaw basin accumulates over a period of years, it will provide a solid basis for improved basin modeling and for upgrading the flood warning program.

Rainfall Data

As previously noted, only one National Weather Service precipitation gage is located within the Squaw basin, and it is near the southern boundary. To obtain realistic measurements of basin-wide rainfall on a time schedule suitable for a flood warning program, it will be necessary to augment the existing rain gage network.

Options Considered There are several technically feasible options for improving rainfall data collection within the basin. As previously reported, automated recording stations that report via telephone or low-power radio are now commonly used in large-scale flood warning programs. Considerable capital expenditure would be necessary to implement an automated network in the Squaw basin.

Radar can also be used to estimate point rainfall intensities and total precipitation depths over a wide area. To date, however, it appears that radar is not widely used for flood prediction in the United States.

High cost and limited accuracy have been the primary problems with radar in the past. In the early 1970s, Grayman and Eagleson (1971 and 1972) studied optimal precipitation networks for flood forecasting and concluded that rain gage networks result in higher net economic benefits than raingage-calibrated radar systems.

In their literature survey of weather radar systems, Bell and James (1985) report, however, that a 1983 cost/benefit study of raingage-calibrated radar networks indicated that these systems could provide net positive economic returns if implemented on a national scale.

A more serious drawback to use of radar systems for flood prediction is lack of accuracy. In their summary of radar measurement of rainfall, Wilson and Brandes (1979) point out that radar estimates of areal and point rainfall are often in error by a factor of two or more. These errors are caused by inaccurate measurement of radar reflectivity, evaporation or advection of rain before it reaches the ground, and variations in drop size distribution.

Errors can be reduced substantially by calibrating radar systems with real-time data from rain gages. When rainfall estimates must be within 30 percent of true values, however, the advantage of raingage-calibrated radar are greatly reduced since the rain gage density required for calibration is sufficient to provide the desired accuracy without the use of radar.

Improved precipitation measurement techniques using radar are currently being studied. Bell and James (1985) report plans to use raingage-calibrated radar, in conjunction with the Texas A&M Watershed Model, for flood forecasting and reservoir operation in the Colorado River Basin. Krajewski and Georgakakos (1985) also report work on generating synthetic radar-rainfall data for testing various methods of merging rain gage and radar data to obtain estimates of mean areal rainfall in large basins. With continuing work, radar rainfall measurement will probably become more widely used in future flood prediction programs. In view of the costs and complexity of setting up a raingage-calibrated radar system, however, such a system could not be realistically considered for use in the proposed Ames Flood Warning Program at the present time.

In view of the strong desire by the city of Ames to keep costs to a minimum, a small network of rainfall observers, using manually read gages, is probably the most feasible rainfall data collection option.

Observers would be asked to maintain daily precipitation records during the period from May through September when the risk of flooding is greatest. During flood watch periods, observers should report accumulated rainfall at six-hour intervals. To facilitate definition of antecedent moisture conditions throughout the basin, observers should also report five-day rainfall totals for the period prior to the storm.

Flood watch periods can be triggered by Weather Service storm forecasts, or they may be defined operationally using rainfall data. Since critical stream stages in Ames are approached as flows reach 3,000 to 4,000 cfs, it is recommended that a flood watch be initiated whenever three or more inches of rainfall occurs under AMC I conditions, or when an inch or more of precipitation occurs under AMC II or III conditions.

When flood watch conditions do not occur simultaneously at all rainfall stations, the flood forecaster on duty in Ames should alert observers in those areas where critical conditions have not developed so that all rainfall in the basin is reported during flood watch periods. The flood forecaster should also be responsible for terminating a flood watch if less than one inch of rainfall occurs during a 48-hour period after a flood watch is initiated.

Rain Gage Network Design One of the most important considerations in planning a rain gage network is the number of gages needed to obtain the desired accuracy in estimation of mean areal rainfall. A brief review of literature reveals surprisingly little general guidance for selecting the number or location of gages in a basin-wide network. It may be that the number of factors that must be considered preclude use of general guidelines. Factors such as basin size, type and duration of rainfall event (thunderstorm versus frontal development), season of the year, prevailing patterns of flood-producing storms (single versus multi-cell storms), orientation of basin with respect to the major axis of large storms, purpose of the rainfall data, and available funding, can all significantly affect network design.

The Field Manual for Research in Agricultural Hydrology

(U.S. Dept. of Agriculture 1979, p. 8) was the only reference located which offers general guidance on gage density (for research networks). Since accuracy is a key concern for research activities, these recommendations are at the upper end of the cost and gage density spectrum. For a 200 square mile basin (roughly equivalent to the Squaw basin) approximately 75 gages are recommended.

Some of the most useful information applicable to raingage network design in the Midwest comes from basin-scale studies conducted by Huff and others from the Illinois Water Survey during the late 1950s and 1960s. Huff and Semonin (1960) studied the duration, orientation, depth, and timing of nearly 300 flood-producing storms that occurred in Illinois between 1914 and 1957. Their analysis showed that over 50 percent of flood-producing storms happen during the summer months of June through August, and that more than 70 percent occur during the six-month period from June through November. These results agree well with data for the Squaw basin (Figure 5).

The Huff and Semonin studies also revealed that the major axis of a flood-producing storm is most frequently oriented along a line running from WSW to ENE, or from WNW to ESE. Diurnal rainfall distributions also indicate a strong tendency for major storms to occur at night.

Later studies by Huff (1970), and Huff and Schickedanz

(1972) showed the effects of raingage network density (square miles per gage), storm duration, basin area, mean areal precipitation, and the type of storm event, on average error in estimating areal rainfall. In general, storm sampling error increased with increasing mean areal precipitation and with decreasing gage density. As storm duration increases, sampling error tends to be reduced. Warm season rain showers and thunderstorms were found to require greater gage density to achieve accurate rainfall estimates than are required for steady rain events.

Based on his studies of basins in the 400 to 550 square mile size range, Huff (1970, p. 37) was able to characterize the mean error in areal rainfall estimation using the following logarithmic expression:

$$\log E = -1.5069 + 0.65 \log P + 0.82 \log G - 0.22 \log T - 0.45 \log A \qquad (17)$$

where:

E = Average error in estimate of mean areal rainfall (inches)

P = Best estimate of areal mean rainfall (inches) based on maximum number of gages in the basin

G = Gage density (square miles per gage)

T = Storm duration (hours)

A = Total basin area (square miles)

Since this relationship reflects midwestern storm characteristics, it was used to help define the number of gages that would be needed to achieve the desired accuracy in estimating mean rainfall for storms in the Squaw basin.

The first step in determining gage spacing was to assess the impacts of basin-wide rainfall errors on predicted flood crests in Ames. To do this several assumed levels of basin-wide average rainfall were input to the flood forecasting aid, and the resulting peak flows at Ames were tabulated. The stage-discharge table for the Squaw Creek gaging station at Lincoln Way was used to determine the approximate flood stage associated with each storm.

This process was repeated assuming mean areal rainfall errors of five and ten percent. The erroneous basin-wide averages were input to the model and the predicted peak flow and stage were determined as before. Since basin response changes considerably with increasing soil moisture, trials were made using both AMC II and AMC III conditions.

Table 24 summarizes the results of this procedure. As indicated by the errors in peak stage, precipitation errors of $\pm$ 10 percent generally cause errors in peak stage estimates of 0.5 feet or more. When precipitation error is less than $\pm$ 5 percent, however, errors in the peak stage are generally less than 0.5 feet.

Since the topography in the Squaw Creek flood plain is relatively flat, it is believed that stage estimates that are

Table 24. Errors in estimated peak flow and stage at Ames associated with five and ten percent errors in estimated mean areal rainfall

"True" Rainfall (inches)	Estimated Rainfall (inches)	Percent Error in Rainfall	Peak Flow (cfs)	Peak Stage (feet)	Stage Error (feet)
AMC II Conditions					
1.50	--	0	2900	8.90	0
	1.65	+10	3600	9.89	+0.99
	1.35	-10	2268	7.57	-1.33
	1.58	+5	3246	9.43	+0.53
	1.43	-5	2576	8.26	-0.64
2.00	--	0	5440	11.62	0
	2.20	+10	6606	12.23	+0.61
	1.80	-10	4356	10.79	-0.83
	2.10	+5	6018	11.93	+0.31
	1.90	-5	4886	11.30	-0.32
2.50	--	0	8476	13.05	0
	2.75	+10	10130	13.67	+0.62
	2.25	-10	6910	12.37	-0.68
	2.62	+5	9294	13.37	+0.32
	2.37	-5	7684	12.70	-0.35
AMC III Conditions					
1.00	--	0	3720	10.04	0
	1.10	+10	4392	10.83	+0.79
	0.90	-10	3086	9.21	-0.83
	1.05	+5	4054	10.44	+0.40
	0.95	-5	3400	9.63	-0.41
1.50	--	0	7324	12.56	0
	1.65	+10	8502	13.06	0.50
	1.35	-10	6186	12.02	-0.54
	1.58	+5	7912	12.82	+0.26
	1.42	-5	6750	12.30	-0.26
2.00	--	0	11360	14.09	0
	2.20	+10	13046	a	
	1.80	-10	9710	13.52	-0.57
	2.10	+5	12200	a	
	1.90	-5	10532	13.81	-0.28

[a]Exceeds range of stage-discharge table.

within 0.5 feet are adequate. Furthermore, since stream valley cross sections at most locations are much wider than at the gaging station, stage at these locations is less sensitive to changes in peak flow than at the gaging station. When considered in light of these facts, the results from Table 24 suggest that if mean areal rainfall errors can be limited to five percent or less, then errors in estimating peak stage will be acceptable.

To determine the gage density necessary to keep the mean error in basin-wide precipitation estimates below five percent, equation 17 was applied using the total basin area (227 square miles) and a storm duration of one hour. This particular storm duration was selected since error decreases with increasing duration as indicated by equation 17.

Figure 77 shows the trend in error for mean areal rainfall estimates as gage density and precipitation quantities are varied. Small storms of one inch or less pose the most critical case. To keep errors within five percent for this type of storm, gage density must be limited to 35 square miles per gage or less. To achieve this, a minimum of seven evenly spaced gages will be needed within the basin. Their exact locations will depend, to some extent, on the availability of rainfall observers and on the proximity to basin boundaries. Since it is desirable to have each observation station roughly in the center of the area it

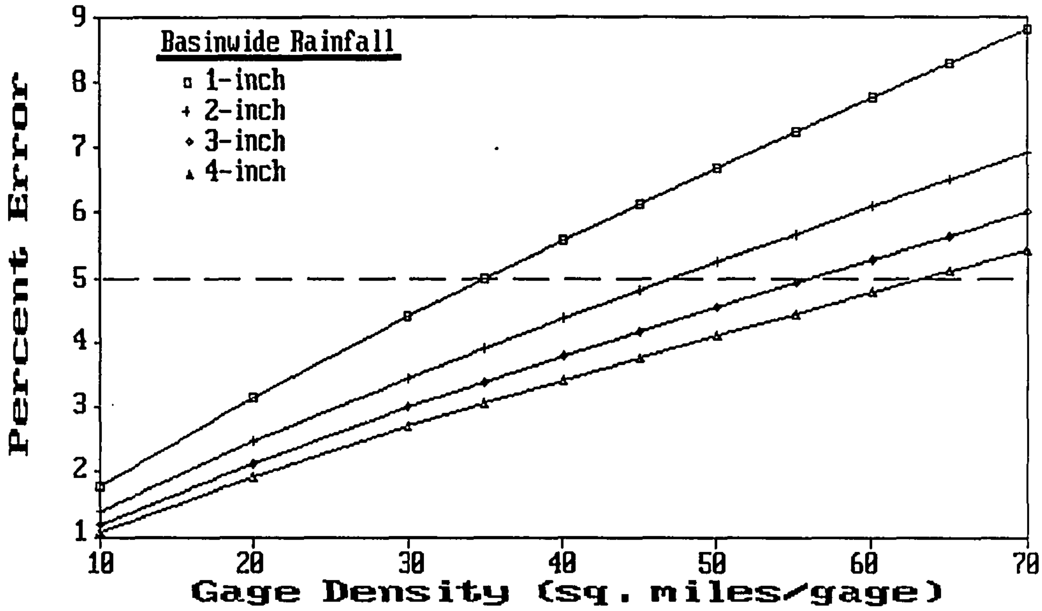

Figure 77. Percent error for estimates of mean areal rainfall for various levels of precipitation

represents, it will be important to locate gages away from the edge of the basin.

Streamflow Data

Knowledge of the streamflow occurring prior to a flood-producing storm is essential to accurate prediction of peak flow caused by the storm. Both the quantity and trend of pre-storm flow must be known since the forecasting aid must project this flow into the future and add the predicted storm contribution to it.

In its current form, the forecasting aid requires two streamflow measurements, taken several hours apart, prior to the flood-producing storm. Equation 13 is solved for the recession constant given two streamflow readings and their time of occurrence. The recession constant is then used to extend the pre-storm flow into the future.

This scheme will work only when pre-storm streamflow is in a gradual recession mode. If flow prior to the storm is rising or falling rapidly, as might be the case when two consecutive storms occur, there is no reliable way to extend the pre-storm record to serve as a base for runoff from a second storm. In cases such as these, it will be necessary to begin flood prediction with streamflow occurring prior to the first rainfall. This will allow use of pre-storm flow data that is in a slow recession mode and permit the model to use all available rainfall data to develop a series of sequential hydrographs.

Since the forecasting aid is capable of handling up to five consecutive storms, this approach should be workable, but it will require careful observation of streamflow during the flood season. During periods of no rainfall, it is recommended that a gage reading be recorded at least once every 24 hours. At times when heavy rainfall is predicted in the basin, streamflow measurements should be recorded at six-hour intervals to insure that usable pre-storm data is available if a major storm materializes. Since stream stage information is readily available through a telephone link with the gaging station at Lincoln Way, this data collection effort will require minimal time commitment by city personnel.

Program Operations and Personnel

Program operations during a flood watch require rainfall observers to measure and report storm rainfall throughout the basin; forecasters, to analyze rainfall and streamflow data, predict peak flow and initiate a flood warning; and communications personnel to alert the public to the predicted flood hazard.

Specific program operations and personnel requirements are summarized below. For the most part, involvement in the flood warning program will not represent a full-time job commitment. It is anticipated that city personnel will handle occasional flood program duties as part of their regular duties.

Program Coordination

The planning, communication, and evaluation needed for a successful flood warning program will require a coordinator to oversee these duties.

Maintaining communication with the National Weather Service, the Office of Disaster Services, local law enforcement officials, and the public will be an important part of the coordinator's duties. Operating procedures must be planned and clearly conveyed to all parties so that each understands their role in the program.

Public education will also be particularly important.

Newspaper articles and radio or television spots should be prepared at the beginning of each flood season to remind citizens of how the flood warning program works, appropriate emergency flood damage prevention measures, and who to contact for further information on the program. Major business which may be affected by flooding should also be contacted and advised of flood program operations.

The flood coordinator should also be in charge of maintaining the rainfall observation and reporting network. This will involve selection and training of the observers, and proper installation and periodic inspection of the rain gaging equipment.

Flood Forecasting

A flood forecaster will need to be on duty at all times during a flood watch period to receive incoming data, update flow forecasts, and issue a flood warning if needed. In the case of prolonged storms several trained individuals may be needed to handle this duty over a span of several 8-hour shifts.

Although the forecasting aid will eliminate the need to make lengthy computations or apply hydrologic theory, it will be essential that all forecasters receive training in maintaining rainfall and streamflow records, using the forecasting aid, and general flood-watch operating procedures.

Since the flood season lasts only about six months of each year, it will be particularly important to give refresher training to forecasters at the beginning of each flood season. Occasional flood warning drills should be scheduled to maintain a state of readiness during the flood season.

Rainfall Observation

Rainfall observers will be responsible for keeping daily rainfall records and for reporting rainfall at six-hour intervals during a flood watch. A flood watch would automatically be initiated anytime three or more inches of rain occurs under AMC I conditions, or when an inch or more of rain occurs under AMC II conditions. In instances of heavy localized rainfall, all observers would be contacted and asked to report at six-hour intervals even though the rain at their location does not meet the criteria for initiating a flood watch. This would insure that basin-wide runoff is taken into account when predicting the flood crest even though subbasins receiving only light rainfall will not contribute heavily to the flood hydrograph.

Communications

In the event that a flood warning is issued, this will need to be communicated as rapidly as possible to agencies, businesses, and homeowners so as to maximize the time available for implementing emergency flood damage control efforts.

It is difficult to anticipate the exact number of groups that will need to be alerted. It is not hard to imagine, however, that the contact list could easily include 50 or more entries. Clearly, the National Weather Service, Story County Office of Disaster Services, and local law enforcement officials must be notified. If important traffic routes are expected to become blocked by flood waters, fire and paramedic teams should be advised of this.

In addition, shift supervisors for city sewer, water, and electric utilities, and traffic control operations, should be informed of impending flooding. This will help them to protect especially vulnerable facilities or to check flood damage control equipment to make sure that it is ready to function. A good example of this would be the metal flap gates on storm sewers that penetrate the levee on the south side of College Creek or along Elwood Drive. These gates have often been found to be wedged open by debris from the parking lots that they drain. A quick check of these gates prior to arrival of a flood could prevent serious backflooding through these storm sewer facilities.

Conveying a flood warning to businesses along South Duff Avenue is particularly important. This is an area that received considerable damage during the flood in 1975, and according to Dougal's written observations (1975), this primarily commercial zone could benefit greatly from advanced flood warning.

Since storm-producing floods often occur during non-business hours, however, special operating provisions will be needed to insure that business owners get the flood warning as soon as possible. According to city maps of the commercial area along South Duff, there are approximately 25 buildings in this area that could be damaged by flooding. It is recommended that merchants in this area be surveyed to see which of them would prefer to be notified in the event a flood warning is issued. Those wishing to be notified should be asked to provide the name and phone number of a person who can be reached, day or night, to receive a flood warning.

The number of residences in the flood plain precludes direct telephone contact as a method of disseminating a flood warning to the general public. Mass warning techniques will need to be used to alert flood-plain dwellers to impending flooding. As previously noted in the review of literature, multiple warning modes are recommended for maximizing human response to a warning (Tamminga 1980). This is particularly true in communities, like Ames, that are not flooded frequently (Day et al. 1969).

Communication modes that could be used in Ames include local radio and television stations. This communications mode is particularly effective in delivering specific information about the level and timing of expected flooding.

The local network of warning sirens should also be considered for flood warning if the sirens near to the flood plain can be activated separately from other sirens throughout the city.

As in the past, law enforcement officers and their vehicles will probably provide the primary communications link with the general public located in the flood plain. Using mobile loud speaker systems, law enforcement officials can quickly alert residential areas to oncoming floods and remind residents of appropriate emergency measures. Since radio and television stations can be knocked off the air during severe storms, mobile communication facilities provided by law enforcement groups must be considered the primary communication channel between the local flood forecaster and residential areas in the flood plain.

Since forecasting personnel will be occupied with receiving rainfall data, monitoring streamflow, and updating the flood forecast, it is recommended that a separate team be assigned to disseminate and receive communications during a flood watch. Police or fire dispatchers, or similar personnel might be considered for this. As with all other flood warning personnel, the duties of communications staff should be clearly outlined in writing and reviewed with the communications team at the beginning of each flood season.

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APPENDIX

Note: Longer lines have been truncated and continued on next line to improve readability.

```
10   LPRINT CHR$(15);
20   LPRINT CHR$(27);"0";
30   CLEAR
40   CLS
50   WIDTH "LPT1:",132
60   DIM FLOSUM%(15,240), NNAME$(13), DAT(13,10,5), T(10),
     DAYS(12)
70   DIM BF%(2,4), D%(10), M%(10), QMAX%(14), MARK%(13),
     CUMP(13), CUMSRO(13)
80   REM FUNCTION TIME(X) CONVERTS 24 HR CLOCK TIME TO DECIMAL
     HOURS
90   REM IE TIME 2230=22.5 HOURS (FOR PURPOSES OF CALCULATING
     TIME OF PEAK)
100 DEF FNTIME(X)=100*(X/100-INT((X+0.0001)/100))/60 +
     INT((X+0.0001)/100)
110 DEF FNT24(X)=INT(X)*100 + (X-INT(X))*60
120 DEF  FNTDIF1(M2,D2,H2,M1,D1,H1) =
     (M2-M1)*(24*(DAYS(M1)-D1+D2-1)) + (M1+1-M2) * (24*(
     D2-(D1+1))) + 24-FNTIME(H1)+FNTIME(H2)
130 ST%=1
140 LPRINT "RUN DATE: ";DATE$, "TIME:  ";TIME$
150 INPUT"DO YOU WISH TO ADD MORE STORM DATA TO A PREVIOUS
     STORM SEQUENCE? (Y OR N)";ANSWER$
160 CLS: IF ANSWER$="Y" OR ANSWER$="y" THEN GOSUB 7340
170 PRINT: PRINT
180 LPRINT:LPRINT
190 LPRINT "STORM  =";ST%
200 INPUT "SUMMARY OUTPUT (1), OR COMPLETE (2)";OUTP%
210 FOR KK = 1 TO 13
220 READ NNAME$(KK)
230 NEXT KK
240 FOR JJ=1 TO 12
250 READ DAYS(JJ)
260 NEXT JJ
270 REM ***** ENTER PRECIP DATA *****
280 GOSUB 6210
290 IF ST%>1 THEN 360
300 REM ************** DETERMINE TIME BASE ****************
310 GOSUB 3670
320 LPRINT "TIME BASE : ",MO%"/"DAY%"     "TBASE
330 REM ***** CALCULATE BASEFLOW FROM FLOW DATA PRIOR TO
     STORM
340 GOSUB 5090
```

```
350 QMAX%(14)=FLOSUM%(15,1)
360 FOR KK7 = 1 TO 13
370 IF KK7=1 THEN RESTORE 750: GOTO 490
380 IF KK7=2 THEN RESTORE 980: GOTO 490
390 IF KK7=3 THEN RESTORE 1240: GOTO 490
400 IF KK7=4 THEN RESTORE 1440: GOTO 490
410 IF KK7=5 THEN RESTORE 1570:GOTO 490
420 IF KK7=6 THEN RESTORE 1700: GOTO 490
430 IF KK7=7 THEN RESTORE 1940: GOTO 490
440 IF KK7=8 THEN RESTORE 2080: GOTO 490
450 IF KK7=9 THEN RESTORE 2250: GOTO 490
460 IF KK7=10 THEN RESTORE 2360: GOTO 490
470 IF KK7=11 THEN RESTORE 2510: GOTO 490
480 IF KK7=12 THEN RESTORE 2760: GOTO 490
485 IF KK7=13 THEN RESTORE 2960: GOTO 490
490 REM ***** DETERMINE SCS CURVE NUMBER FOR SUBBASIN *****
500 GOSUB 3820
510 REM ***** CALC RUNOFF USING SCS METHOD *****
520 GOSUB 3920
530 REM ***** CALC SUBBASIN PEAK FLOW CONTRIBUTION @
    GAGE*****
540 GOSUB 4000
550 REM ***** CALC ARRIVAL TIME FOR SUBBASIN PEAK FLOW @
    GAGE*****
560 GOSUB 4050
570 REM ***** SUPERIMPOSE FLOWS FROM EACH SUBBASIN AND
    SUM*****
580 GOSUB 4560
590 NEXT KK7
650 INPUT"DO YOU WISH TO ENTER ANOTHER STORM? (Y OR
    N)";ANSWER$
660 CLS: IF ANSWER$ = "N" OR ANSWER$="n" THEN 700
670 ST%= ST% + 1
680 LPRINT "STORM # =";ST%
690 GOTO 270
700 GOSUB 7170
710 DATA A, B1,B2,C1,C2,D1,D2,D3,D4
720 DATA E1,E2,F,G
730 DATA 31,28,31,30,31,30,31,31,30,31,30,31
740 REM ******SUBBASIN A
750 DATA 64, 81, 92, 1317, 21.00, 39, 0.086
760 DATA    0.1,  0.1,  0.2,  0.3,  0.4,  0.6,  0.9
770 DATA    1.3,  1.8,  2.5,  3.3,  4.4,  5.7,  7.4
780 DATA    9.3, 11.6, 14.2, 17.3, 20.7, 24.5, 28.7
790 DATA   33.2, 38.0, 43.1, 48.3, 53.7, 59.1, 64.4
800 DATA   69.6, 74.6, 79.3, 83.7, 87.6, 91.1, 94.0
810 DATA   96.3, 98.1, 99.3, 99.9,100.0, 99.5, 98.5
820 DATA   97.1, 95.2, 92.9, 90.2, 87.3, 84.1, 80.7
830 DATA   77.1, 73.5, 69.8, 66.1, 62.4, 58.8, 55.3
840 DATA   51.9, 48.6, 45.5, 42.6, 39.9, 37.3, 34.8
```

```
850 DATA   32.6, 30.4, 28.5, 26.6, 24.9, 23.3, 21.7
860 DATA   20.3, 19.0, 17.8, 16.6, 15.5, 14.5, 13.6
870 DATA   12.7, 11.9, 11.1, 10.4,  9.7,  9.0,  8.4
880 DATA    7.9,  7.4,  6.9,  6.4,  6.0,  5.6,  5.3
890 DATA    4.9,  4.6,  4.3,  4.0,  3.8,  3.5,  3.3
900 DATA    3.1,  2.9,  2.7,  2.5,  2.3,  2.2,  2.1
910 DATA    1.9,  1.8,  1.7,  1.6,  1.5,  1.4,  1.3
920 DATA    1.2,  1.1,  1.1,  1.0,  0.9,  0.9,  0.8
930 DATA    0.8,  0.7,  0.6,  0.6,  0.5,  0.5,  0.4
940 DATA    0.4,  0.4,  0.3,  0.3,  0.2,  0.2,  0.2
950 DATA    0.2,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1
960 DATA    0.1,  0.1,  0.0
970 REM ******SUBBASIN B1
980 DATA 64, 81, 92, 958, 21.25, 45, 0.079
990 DATA    0.1,  0.1,  0.1,  0.2,  0.3,  0.5,  0.7
1000 DATA    1.1,  1.5,  2.0,  2.6,  3.5,  4.5,  5.7
1010 DATA    7.1,  8.7, 10.6, 12.8, 15.2, 17.9, 20.9
1020 DATA   24.1, 27.6, 31.3, 35.3, 39.5, 43.8, 48.2
1030 DATA   52.8, 57.3, 61.9, 66.3, 70.7, 74.9, 78.8
1040 DATA   82.5, 85.9, 88.9, 91.6, 94.0, 95.9, 97.5
1050 DATA   98.7, 99.5, 99.9,100.0, 99.8, 99.2, 98.3
1060 DATA   97.2, 95.7, 94.1, 92.2, 90.1, 87.8, 85.3
1070 DATA   82.7, 80.0, 77.2, 74.2, 71.3, 68.3, 65.3
1080 DATA   62.3, 59.4, 56.5, 53.7, 51.0, 48.4, 46.0
1090 DATA   43.6, 41.4, 39.3, 37.3, 35.4, 33.6, 32.0
1100 DATA   30.4, 28.9, 27.5, 26.2, 24.9, 23.7, 22.5
1110 DATA   21.4, 20.4, 19.4, 18.4, 17.5, 16.7, 15.8
1120 DATA   15.0, 14.3, 13.6, 12.9, 12.3, 11.7, 11.1
1130 DATA   10.5, 10.0,  9.5,  9.0,  8.6,  8.2,  7.8
1140 DATA    7.4,  7.0,  6.7,  6.3,  6.0,  5.7,  5.4
1150 DATA    5.2,  4.9,  4.7,  4.4,  4.2,  4.0,  3.8
1160 DATA    3.6,  3.4,  3.3,  3.1,  3.0,  2.8,  2.7
1170 DATA    2.5,  2.4,  2.3,  2.2,  2.1,  2.0,  1.9
1180 DATA    1.8,  1.7,  1.6,  1.5,  1.5,  1.4,  1.3
1190 DATA    1.3,  1.2,  1.2,  1.1,  1.1,  1.0,  1.0
1200 DATA    0.9,  0.9,  0.8,  0.8,  0.7,  0.7,  0.6,
1210 DATA    0.6,  0.5,  0.5,  0.4,  0.4,  0.3,  0.3,
1220 DATA    0.2,  0.2,  0.1,  0.1,  0
1230 REM ******SUBBASIN B2
1240 DATA   64, 81, 92, 1920, 18.25, 34, 0.108
1250 DATA    0.1,  0.1,  0.2,  0.3,  0.5,  0.8,  1.2
1260 DATA    1.7,  2.4,  3.4,  4.6,  6.2,  8.1, 10.5
1270 DATA   13.3, 16.6, 20.4, 24.8, 29.5, 34.7, 40.3
1280 DATA   46.2, 52.3, 58.4, 64.5, 70.5, 76.1, 81.4
1290 DATA   86.1, 90.3, 93.7, 96.5, 98.4, 99.6,100.0
1300 DATA   99.7, 98.6, 96.9, 94.6, 91.8, 88.5, 84.9
1310 DATA   81.0, 76.9, 72.6, 68.3, 64.0, 59.8, 55.7
1320 DATA   51.7, 48.0, 44.4, 41.1, 37.9, 35.0, 32.3
1330 DATA   29.8, 27.5, 25.4, 23.4, 21.6, 19.9, 18.4
1340 DATA   17.0, 15.6, 14.4, 13.3, 12.3, 11.3, 10.4
```

```
1350 DATA    9.6,  8.9,  8.2,  7.5,  6.9,  6.4,  5.9
1360 DATA    5.4,  5.0,  4.6,  4.3,  3.9,  3.6,  3.4
1370 DATA    3.1,  2.9,  2.6,  2.4,  2.2,  2.1,  1.9
1380 DATA    1.8,  1.6,  1.5,  1.4,  1.3,  1.2,  1.1
1390 DATA    1.0,  0.9,  0.9,  0.8,  0.7,  0.7,  0.6
1400 DATA    0.5,  0.5,  0.4,  0.4,  0.3,  0.3,  0.2
1410 DATA    0.2,  0.2,  0.1,  0.1,  0.1,  0.1,  0.1
1420 DATA    0.1,  0.0
1430 REM ******SUBBASIN C1
1440 DATA   60, 78, 90, 2075, 13.25, 26, 0.079
1450 DATA   0.1,  0.1,  0.3,  0.5
1460 DATA   0.8, 1.3,  2.0,  3.1,  4.7,  6.8,  9.6, 13.1
1470 DATA   17.5, 22.8, 29.0, 35.9, 43.5, 51.6, 59.9, 68.1
1480 DATA   76.0,  83.1,  89.2, 94.1, 97.6, 99.6, 100.0, 98.9
1490 DATA  96.5, 92.9,  88.3, 82.9, 77.0, 70.8, 64.5, 58.4
1500 DATA  52.4, 46.8, 41.6, 36.8, 32.4, 28.5, 25.1,  22.0
1510 DATA  19.2, 16.8, 14.7, 12.9, 11.3,  9.9,  8.6,  7.5
1520 DATA  6.6,  5.8,  5.0,  4.4,  3.9,   3.4,  2.9,  2.6
1530 DATA  2.3,  2.0,  1.7,  1.5,  1.3,  1.1,  1.0,  0.8
1540 DATA  0.7,  0.6,  0.5,  0.4,  0.3,  0.3,  0.2,  0.2
1550 DATA 0.1,   0.1,   0.1,   0.1,   0.1,  0
1560 REM ******SUBBASIN C2
1570 DATA 63, 80, 91, 1726, 13.50, 26, 0.068
1580 DATA  0.1,  0.1,  0.3,  0.4
1590 DATA  0.8,  1.2,  2.0,  3.0,  4.5,  6.4,  9.0, 12.3
1600 DATA 16.5, 21.4, 27.2, 33.7, 40.9, 48.6, 56.7, 64.7
1610 DATA 72.5, 79.8, 86.3, 91.7, 95.8, 98.6, 100.0, 99.9
1620 DATA 98.5, 95.8, 92.1, 87.5, 82.2, 76.5, 70.6, 64.5
1630 DATA 58.6,  52.9,  47.5, 42.4, 37.8, 33.6, 29.8, 26.3
1640 DATA 23.3, 20.5, 18.1, 16.0, 14.1, 12.4, 11.0,  9.7
1650 DATA  8.5,  7.5,  6.6,  5.8,  5.1,  4.5,  4.0,  3.5
1660 DATA  3.1,  2.7,  2.4,  2.1,  1.9,  1.7,  1.5,  1.3
1670 DATA  1.1,  1.0,  0.9,  0.7,  0.6,  0.5,  0.4
1680 DATA  0.3, 0.2, 0.2, 0.2, 0.1, 0.1, 0.1, 0.1, 0.1, 0
1690 REM ******SUBBASIN D1
1700 DATA 62, 79, 91, 1222, 17.00, 47, 0.116
1710 DATA  0.1, 0.1, 0.2, 0.3, 0.5
1720 DATA  0.7,  1.0,  1.5,  2.0,  2.7,  3.6,  4.6,  5.9
1730 DATA  7.3,  8.9, 10.7, 12.8, 15.0, 17.4, 20.1, 22.9
1740 DATA 26.0, 29.2, 32.7, 36.3, 40.1, 44.1, 48.2, 52.3
1750 DATA 56.5, 60.7, 64.9, 68.9, 72.8, 76.6, 80.1, 83.3
1760 DATA 86.3, 89.0, 91.4, 93.5, 95.3, 96.8, 98.0, 98.9
1770 DATA 99.5, 99.9, 100.0, 99.9, 99.5, 98.9, 98.1, 97.1
1780 DATA 95.9, 94.5, 93.0, 91.3, 89.5, 87.6, 85.6, 83.4
1790 DATA 81.2, 78.8, 76.4, 73.9, 71.3, 68.6, 66.0, 63.3
1800 DATA 60.6, 58.0, 55.4, 53.0, 50.6, 48.3, 46.1, 44.1
1810 DATA 42.1, 40.3, 38.5, 36.9, 35.3, 33.8, 32.4, 31.1
1820 DATA 29.8, 28.6, 27.5, 26.4, 25.4, 24.3, 23.4, 22.4
1830 DATA 21.5, 20.7, 19.8, 19.0, 18.2, 17.4, 16.7, 16.0
1840 DATA 15.3, 14.7, 14.1, 13.5, 12.9, 12.4, 11.9, 11.4
```

```
1850 DATA 10.9, 10.5, 10.0,  9.6,  9.2,  8.8,  8.5,  8.1
1860 DATA  7.8,  7.5,  7.1,  6.8,  6.5,  6.3,  6.0,  5.8
1870 DATA  5.5,  5.3,  5.1,  4.9,  4.7,  4.5,  4.3,  4.1
1880 DATA  3.9,  3.8,  3.6,  3.5,  3.3,  3.2,  3.1,  2.9
1890 DATA  2.8,  2.7,  2.6,  2.5,  2.4,  2.3,  2.2,  2.1
1900 DATA  2.0,  1.9,  1.8,  1.7,  1.6,  1.4,  1.3,  1.2
1910 DATA  1.0,  0.9,  0.7,  0.6,  0.5,  0.4,  0.3,  0.2
1920 DATA  0.2,  0.1,  0.1,  0
1930 REM ******SUBBASIN D2
1940 DATA 63, 80, 91, 1686, 11.00, 26, 0.070
1950 DATA 0.1,  0.2,  0.3
1960 DATA  0.5,  0.9,  1.5,  2.4,  3.7,  5.5,  7.9, 11.1
1970 DATA 15.0, 19.7, 25.3, 31.7, 38.9, 46.5, 54.5, 62.5
1980 DATA 70.4, 77.8, 84.4, 90.1, 94.5, 97.7, 99.5, 100.0
1990 DATA 99.2, 97.1, 94.0, 90.0, 85.3, 80.1, 74.5, 68.8
2000 DATA 63.1, 57.5, 52.1, 47.1, 42.5, 38.2, 34.3, 30.8
2010 DATA 27.6, 24.8, 22.2, 19.9, 17.9, 16.0, 14.4, 12.9
2020 DATA 11.5, 10.3,  9.3,  8.3,  7.4,  6.7,  6.0,  5.4
2030 DATA  4.8,  4.3,  3.9,  3.5,  3.1,  2.8,  2.5,  2.3
2040 DATA  2.0,  1.8,  1.6,  1.5,  1.3,  1.2,  1.1,  1.0
2050 DATA  0.8,  0.7,  0.7,  0.6,  0.5,  0.4,  0.3,  0.3
2060 DATA  0.2, 0.2,  0.1,  0.1,  0.1,  0.1,  0.1,  0
2070 REM ******SUBBASIN D3
2080 DATA 63, 80, 91, 832, 12.50, 30, 0.045
2090 DATA  0.1,  0.2,  0.3
2100 DATA  0.5,  0.8,  1.3,  2.0,  3.0,  4.3,  6.1,  8.3
2110 DATA 11.0, 14.2, 18.1, 22.6, 27.6, 33.2, 39.2, 45.6
2120 DATA 52.2, 58.8, 65.5, 71.8, 77.8, 83.3, 88.1, 92.2
2130 DATA 95.4, 97.8, 99.3, 100.0, 99.9, 98.9, 97.3, 95.0
2140 DATA 92.2, 88.9, 85.2, 81.1, 76.8, 72.4, 67.9, 63.4
2150 DATA 58.9, 54.7, 50.6, 46.8, 43.2, 39.8, 36.7, 33.9
2160 DATA 31.3, 28.9, 26.7, 24.7, 22.8, 21.1, 19.5, 18.0
2170 DATA 16.7, 15.4, 14.2, 13.1, 12.1, 11.2, 10.3,  9.5
2180 DATA  8.8,  8.1,  7.5,  6.9,  6.4,  5.9,  5.5,  5.0
2190 DATA  4.7,  4.3,  4.0,  3.7,  3.4,  3.1,  2.9,  2.7
2200 DATA  2.5,  2.3,  2.1,  2.0,  1.8,  1.7,  1.6,  1.4
2210 DATA  1.3,  1.2,  1.2,  1.1,  1.0,  0.9,  0.8,  0.8
2220 DATA  0.7,  0.7,  0.6,  0.5,  0.5,  0.4,  0.4,  0.3
2230 DATA  0.3,  0.2,  0.2,  0.2,  0.1,  0.1,  0.1,  0
2240 REM ******SUBBASIN D4
2250 DATA 62, 79, 91, 2605, 9.75, 22, 0.080
2260 DATA  0.1,  0.2,  0.3
2270 DATA   0.6,  1.2,  2.0,  3.4,  5.4,  8.2, 12.0, 16.9
2280 DATA  23.1, 30.4, 38.8, 48.0, 57.7, 67.4, 76.6, 84.8
2290 DATA  91.6, 96.5, 99.3, 100.0, 98.5, 95.2, 90.2, 84.0
2300 DATA  76.9, 69.4, 61.9, 54.5, 47.6, 41.2, 35.4, 30.3
2310 DATA  25.8, 22.0, 18.6, 15.8, 13.4, 11.3,  9.6,  8.1
2320 DATA   6.8,  5.8,  4.9,  4.1,  3.5,  3.0,  2.5,  2.1
2330 DATA   1.8,  1.5,  1.3,  1.0,  0.9,  0.7,  0.6,  0.5
2340 DATA  0.4,  0.3,  0.2,  0.2,  0.1,  0.1, 0.1,  0
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2350 REM ******SUBBASIN E1
2360 DATA 60, 78, 90, 2625, 6.75, 25, 0.126
2370 DATA  0.1,  0.2,  0.5,  1.0,  1.9,  3.1,  4.8,  7.1
2380 DATA 10.1, 13.9, 18.5, 24.0, 30.2, 37.0, 44.3, 52.0
2390 DATA 59.8, 67.5, 74.8, 81.5, 87.3, 92.1, 95.7, 98.3
2400 DATA 99.7, 100.0, 99.4, 97.9, 95.7, 92.8, 89.2, 85.2
2410 DATA 80.7, 75.9, 71.0, 65.9, 60.9, 55.9, 51.2, 46.7
2420 DATA 42.7, 39.1, 35.8, 32.9, 30.2, 27.8, 25.6, 23.6
2430 DATA 21.7, 20.0, 18.4, 16.9, 15.6, 14.3, 13.1, 12.1
2440 DATA 11.1, 10.2,  9.4,  8.6,  7.9,  7.2,  6.7,  6.1
2450 DATA  5.6,  5.2,  4.7,  4.4,  4.0,  3.7,  3.4,  3.1
2460 DATA  2.9,  2.6,  2.4,  2.2,  2.1,  1.9,  1.7,  1.6
2470 DATA  1.5,  1.4,  1.3,  1.2,  1.1,  1.0,  0.9,  0.9
2480 DATA  0.8,  0.7,  0.7,  0.6,  0.5,  0.5,  0.4,  0.4
2490 DATA  0.3,  0.3,  0.2,  0.2,  0.1,  0.1,  0.1,  0.0
2500 REM ******SUBBASIN E2
2510 DATA 62, 79, 91, 946, 10.25, 38, 0.075
2520 DATA  0.1,  0.1,  0.3,  0.5,  0.9,  1.4,  2.1,  3.1
2530 DATA  4.4,  5.9,  7.8,  9.9, 12.3, 15.0, 18.0, 21.2
2540 DATA 24.8, 28.6, 32.8, 37.2, 41.9, 46.8, 51.9, 57.0
2550 DATA 62.2, 67.3, 72.1, 76.8, 81.0, 84.9, 88.3, 91.3
2560 DATA 93.8, 95.9, 97.5, 98.7, 99.5, 99.9, 100.0, 99.7
2570 DATA 99.1, 98.2, 97.1, 95.6, 94.0, 92.0, 89.9, 87.7
2580 DATA 85.3, 82.7, 80.1, 77.3, 74.3, 71.2, 68.0, 64.8
2590 DATA 61.6, 58.3, 55.2, 52.1, 49.3, 46.6, 44.1, 41.8
2600 DATA 39.6, 37.6, 35.7, 34.0, 32.3, 30.7, 29.3, 27.9
2610 DATA 26.6, 25.4, 24.2, 23.1, 22.0, 20.9, 19.9, 19.0
2620 DATA  18.0, 17.1, 16.3, 15.5, 14.7, 14.0, 13.3, 12.6
2630 DATA  12.0, 11.4, 10.9, 10.4,  9.9,  9.4,  8.9,  8.5
2640 DATA   8.1,  7.6,  7.3,  6.9,  6.6,  6.2,  5.9,  5.6
2650 DATA   5.4,  5.1,  4.8,  4.6,  4.4,  4.2,  4.0,  3.8
2660 DATA   3.6,  3.4,  3.3,  3.1,  3.0,  2.8,  2.7,  2.5
2670 DATA   2.4,  2.3,  2.2,  2.1,  2.0,  1.9,  1.8,  1.7
2680 DATA   1.6,  1.5,  1.5,  1.4,  1.3,  1.3,  1.2,  1.2
2690 DATA   1.1,  1.1,  1.0,  1.0,  0.9,  0.9,  0.9,  0.8
2700 DATA   0.8,  0.7,  0.7,  0.7,  0.6,  0.6,  0.6,  0.5
2710 DATA   0.5,  0.4,  0.3,  0.3,  0.2,  0.2,  0.1,  0.1
2720 DATA   0.1,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1
2730 DATA   0.1,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1
2740 DATA   0.1,  0.0
2750 REM ******SUBBASIN F
2760 DATA 59, 77, 89, 1071, 7.25, 29, 0.067
2770 DATA   0.1,  0.3,  0.7,  1.3,  2.2,  3.4,  5.0,  7.1
2780 DATA   9.7, 12.7, 16.3, 20.3, 24.7, 29.6, 34.9, 40.7
2790 DATA  46.8, 53.1, 59.6, 66.0, 72.1, 77.9, 83.1, 87.6
2800 DATA  91.4, 94.5, 96.8, 98.6, 99.6, 100.0, 99.9, 99.1
2810 DATA  98.0, 96.4, 94.5, 92.2, 89.4, 86.5, 83.3, 79.9
2820 DATA  76.3, 72.5, 68.6, 64.5, 60.5, 56.5, 52.6, 48.9
2830 DATA  45.6, 42.5, 39.7, 37.2, 34.9, 32.8, 30.8, 28.9
2840 DATA  27.2, 25.6, 24.1, 22.7, 21.4, 20.1, 18.9, 17.7
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```
2850 DATA  16.6, 15.6, 14.6, 13.7, 12.8, 12.0, 11.3, 10.6
2860 DATA  10.0,  9.4,  8.8,  8.2,  7.7,  7.2,  6.8,  6.4
2870 DATA   6.0,  5.6,  5.2,  4.9,  4.6,  4.3,  4.1,  3.8
2880 DATA   3.6,  3.4,  3.2,  3.0,  2.8,  2.6,  2.5,  2.3
2890 DATA   2.2,  2.0,  1.9,  1.8,  1.7,  1.6,  1.5,  1.4
2900 DATA   1.3,  1.2,  1.2,  1.1,  1.0,  1.0,  0.9,  0.9
2910 DATA   0.8,  0.8,  0.8,  0.7,  0.7,  0.6,  0.6,  0.5
2920 DATA   0.5,  0.4,  0.4,  0.4,  0.3,  0.3,  0.2,  0.2
2930 DATA   0.2,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1,  0.1
2940 DATA   0.1,  0.1,  0.1,  0.1,  0.1,  0.0
2950 REM *****SUBBASIN G
2960 DATA 59, 77, 89, 1224, 6.50, 26, 0.067
2970 DATA   0.1,  0.4,  0.8,  1.6,  2.6,  4.1,  6.0,  8.6
2980 DATA  11.8, 15.5, 19.9, 24.8, 30.4, 36.4, 42.9, 49.7
2990 DATA  56.8, 63.9, 70.7, 77.2, 83.0, 88.0, 92.1, 95.4
3000 DATA  97.7, 99.3, 100.0, 100.0, 99.3, 98.0, 96.2, 93.9
3010 DATA  91.2, 87.9, 84.4, 80.6, 76.6, 72.4, 68.0, 63.6
3020 DATA  59.1, 54.7, 50.5, 46.6, 43.1, 39.9, 37.0, 34.5
3030 DATA  32.2, 30.0, 28.0, 26.1, 24.4, 22.8, 21.3, 19.9
3040 DATA  18.5, 17.2, 16.0, 14.9, 13.9, 12.9, 12.0, 11.2
3050 DATA  10.5,  9.7,  9.1,  8.4,  7.9,  7.3,  6.8,  6.3
3060 DATA   5.9,  5.5,  5.1,  4.7,  4.4,  4.1,  3.8,  3.6
3070 DATA   3.3,  3.1,  2.9,  2.7,  2.5,  2.3,  2.2,  2.0
3080 DATA   1.9,  1.8,  1.6,  1.5,  1.4,  1.3,  1.2,  1.2
3090 DATA   1.1,  1.0,  1.0,  0.9,  0.9,  0.8,  0.8,  0.7
3100 DATA   0.6,  0.6,  0.5,  0.5,  0.5,  0.4,  0.4,  0.3
3110 DATA   0.3,  0.2,  0.2,  0.1,  0.1,  0.1,  0.0
3120 LPRINT "END OF JOB"
3130 END
3140 REM ***SUBROUTINE TO DISPLAY PRECIP INPUT SUMMARY ******
3150 CLS
3160 PRINT TAB(35) "INPUT DATA SUMMARY FOR STORM # ";ST% :
     PRINT
3170 PRINT TAB(25)"COL#1" TAB(35)"COL#2" TAB(45)"COL#3"
     TAB(55)"COL#4" TAB(65)"COL#5/#6"
3180 PRINT
3190 PRINT TAB(10)"SUBBASIN" TAB(25)"STORM" TAB(35)"5-DAY"
     TAB(45)"TIME" TAB(55)"DURATION"
3200 PRINT TAB(25)"PRECIP"TAB(35)"PRECIP" TAB(45)"BEGAN"
     TAB(55)"HOURS" TAB(65)"MONTH/DAY"
3210 FOR KK2 = 1 TO 13
3220 PRINT"ROW# ";KK2 TAB(12) NNAME$(KK2);TAB(25)
3230 PRINT USING "##.##"; DAT(KK2,1,ST%);
3240 PRINT TAB(35)
3250 PRINT USING "##.##"; DAT(KK2,2,ST%);
3260 PRINT TAB(45)
3270 PRINT USING "####"; DAT(KK2,3,ST%);
3280 PRINT TAB(55)
3290 PRINT USING "##.##"; DAT(KK2,4,ST%);
3300 PRINT TAB(65)
```

```
3310 PRINT USING "   ##/##";DAT(KK2,9,ST%),DAT(KK2,10,ST%)
3320 NEXT KK2
3330 PRINT:PRINT
3340 RETURN
3350 REM **********SUBROUT.TO PRINT PRECIP INPUT SUMMARY ****
3360 CLS
3370 LPRINT TAB(35) "INPUT DATA SUMMARY FOR STORM # ";ST% :
     LPRINT
3380 LPRINT TAB(25)"COL#1" TAB(35)"COL#2" TAB(45)"COL#3"
     TAB(55)"COL#4" TAB(65)"COL#5/#6"
3390 LPRINT
3400 LPRINT TAB(10)"SUBBASIN" TAB(25)"STORM" TAB(35)"5-DAY"
     TAB(45)"TIME" TAB(55)"DURATION"
3410 LPRINT TAB(25)"PRECIP"TAB(35)"PRECIP" TAB(45)"BEGAN"
     TAB(55)"HOURS" TAB(65)"MONTH/DAY"
3420 FOR KK2 = 1 TO 13
3430 LPRINT"ROW# ";KK2 TAB(12) NNAME$(KK2);TAB(25)
3440 LPRINT USING "##.##"; DAT(KK2,1,ST%);
3450 LPRINT TAB(35)
3460 LPRINT USING "##.##"; DAT(KK2,2,ST%);
3470 LPRINT TAB(45)
3480 LPRINT USING "####"; DAT(KK2,3,ST%);
3490 LPRINT TAB(55)
3500 LPRINT USING "##.##"; DAT(KK2,4,ST%);
3510 LPRINT TAB(65)
3520 LPRINT USING "   ##/##";DAT(KK2,9,ST%),DAT(KK2,10,ST%)
3530 NEXT KK2
3540 LPRINT:LPRINT
3550 RETURN
3670 REM ****SUBROUTINE TO DETERMINE TIME BASE**********
3680 MO% = DAT(1,9,ST%)
3690 FOR KK3=2 TO 13
3700 IF DAT(KK3,9,ST%)<MO% THEN MO%=DAT(KK3,9,ST%)
3710 NEXT KK3
3720 DAY% = DAT(1,10,ST%)
3730 FOR KK4=2 TO 13
3740 IF DAT(KK4,9,ST%)=MO% AND DAT(KK4,10,ST%)<DAY% THEN DAY%
     = DAT(KK4,10,ST%)
3750 NEXT KK4
3760 HR% = DAT(1,3,ST%)
3770 FOR KK5 = 2 TO 13
3780 IF DAT(KK5,9,ST%)=MO% AND DAT(KK5,10,ST%)=DAY% AND
     DAT(KK5,3,ST%)<HR% THEN HR%=DAT(KK5,3,ST%)
3790 NEXT KK5
3800 TBASE=HR%
3810 RETURN
3820 REM **** SUBROUTINE TO DETERMINE CURVE NUMBER *****
3830 REM
3840 READ CN1,CN2,CN3
3850 IF DAT(KK7,2,ST%)< 1.4 THEN CN=CN1 ELSE 3870
```

```
3860 GOTO 3900
3870 IF DAT(KK7,2,ST%)> 2.1 THEN CN=CN3 ELSE 3890
3880 GOTO 3900
3890 CN=CN2
3900 DAT(KK7,5,ST%)=CN
3910 RETURN
3920 REM *****SUBROUTINE TO CALCULATE SURFACE RUNOFF *****
3930 S= (1000/CN) - 10
3940 CUMP(KK7)=CUMP(KK7)+DAT(KK7,1,ST%)
3950 SRO=CUMSRO(KK7)
3960 CUMSRO(KK7)=(CUMP(KK7) -(.2*S))^2/(CUMP(KK7) + (.8*S))
3970 SRO=CUMSRO(KK7)-SRO
3980 DAT(KK7,6,ST%)=SRO
3990 RETURN
4000 REM  ***SUBROUTINE TO CALC PEAK FLOW @ GAGE*****
4010 READ QSLOPE
4020 QPK= QSLOPE*SRO
4030 DAT(KK7,7,ST%)=QPK
4040 RETURN
4050 REM**** SUBROUTINE TO CALC TIME OF PEAK FLOW @ GAGE****
4060 REM ALL PEAK FLOW ARRIVAL TIMES REFERENCED FROM TIME OF
     EARLIEST RAINFALL
4070 READ TPK1
4080 DUR=DAT(KK7,4,ST%)
4090 REM ADJUST TIME OF PEAK FOR STORM DURATIONS > 1 HR
4100 DURADJ=(1.5*(DUR^2-(7*DUR)+6)/(-6))+(3.75*(DUR^2
     -(4*DUR)+3)/15)
4110 TPK2= TPK1 + DURADJ
4120 REM ADJUST TIME OF PEAK FOR DIFFERENCES BETWEEN
     SUBBASINS IN STARTING
4130 REM TIME OF STORM
4140 TPK3= TPK2 + FNTDIF1(DAT(KK7,9,ST%), DAT(KK7,10,ST%),
     DAT(KK7,3,ST%), MO%,DAY%,TBASE)
4150 TPK%= TPK3*4
4160 DAT(KK7,8,ST%)= TPK3
4170 RETURN
4560 REM************** SUBROUTINE TO CALC FLOW @ GAGE ****
4570 READ NRISE%
4580 READ FAREA
4590 IF ST%=1 AND TPK%-NRISE%>=1 THEN QMAX%(KK7) =
     FLOSUM%(15,TPK%-NRISE%)*FAREA
4600 IF ST%=1 AND TPK%-NRISE% < 1 THEN QMAX%(KK7) =
     FLOSUM%(15,0)*FAREA
4610 IF ST%>1 THEN FAREA=0
4630 IF TPK%-NRISE%>1 THEN 4690
4640 FOR K8=TPK%-NRISE% TO 0
4650 READ RISE
4660 NEXT K8
4670 K8 = 1
4680 GOTO 4780
```

```
4690 REM+++FILL IN BASEFLOW PRIOR TO SRO
4700 IF ST%>1 THEN K8=TPK%-NRISE%:  GOTO 4780
4710 FOR K8 = 1 TO TPK%-NRISE%-1
4720 INCR= FAREA*FLOSUM%(15,K8)
4730 FLOSUM%(KK7,K8) = INCR + FLOSUM%(KK7,K8)
4740 FLOSUM%(14,K8) = FLOSUM%(14,K8) + INCR/2
4750 IF FLOSUM%(KK7,K8)> QMAX%(KK7) THEN QMAX%(KK7) =
     FLOSUM%(KK7,K8)
4770 NEXT K8
4780 REM+++ADD SRO TO BASEFLOW
4800 CKMARK%=0
4810 PFLAG% = 0
4820 FOR K8 =  K8 TO 240
4830 READ RISE
4840 IF RISE = 100 THEN PFLAG%=PFLAG% + 1
4850 IF RISE=0 THEN 4960
4860 INCR = QPK*RISE/100 + FAREA*FLOSUM%(15,K8)
4870 FLOSUM%(KK7,K8) = INCR + FLOSUM%(KK7,K8)
4880 FLOSUM%(14,K8) = FLOSUM%(14,K8) + INCR/2
4890 IF FLOSUM%(KK7,K8) > QMAX%(KK7) THEN QMAX%(KK7) =
     FLOSUM%(KK7,K8)
4910 IF CKMARK%=1 THEN 4950
4920 IF FLOSUM%(KK7,K8) <= .2*QMAX%(KK7) AND PFLAG%>0 THEN
     CKMARK%=1:MARK%(KK7)=K8
4930 IF K8=240 AND CKMARK%=0 THEN MARK%(KK7)=240
4950 NEXT K8
4960 REM+++CONTINUE WITH FLOW PRECEEDING 1ST STORM
4970 FOR K8 = K8 TO 240
4980 INCR = FAREA*FLOSUM%(15,K8)
4990 FLOSUM%(KK7,K8) = FLOSUM%(KK7,K8) + INCR
5000 FLOSUM%(14,K8) = FLOSUM%(14,K8) + INCR/2
5020 IF FLOSUM%(KK7,K8) <= .2*QMAX%(KK7) AND PFLAG%>0 THEN
     CKMARK%=1:MARK%(KK7)=K8
5030 IF K8=240 AND CKMARK%=0 THEN MARK%(KK7)=240
5060 NEXT K8
5080 RETURN
5090 REM ************ SUBROUTINE TO ESTABLISH BASEFLOW ****
5100 PRINT "**** ENTER TWO POINTS ON RECESSION CURVE PRIOR TO
     RAINFALL ********"
5110 PRINT
5120 GOSUB 5820
5140 FOR J0=1 TO 2
5150 PRINT"ROW";J0;TAB(35)
5170 INPUT;BF%(J0,1)
5180 PRINT TAB(45)
5190 INPUT;BF%(J0,2)
5200 PRINT TAB(55)
5210 INPUT;BF%(J0,3)
5220 PRINT TAB(65)
5230 INPUT BF%(J0,4)
```

```
5240 NEXT J0
5250 CLS
5260 GOSUB 5960
5270 INPUT "ARE INPUT CORRECTIONS NEEDED? (ENTER Y OR
     N)";ANSWER$
5280 IF ANSWER$="N" OR ANSWER$="n" THEN CLS: GOTO 5390
5290 INPUT "ROW#";ROW%
5300 INPUT "COL#";COL%
5310 IF ROW%>2 OR COL%>4 THEN PRINT"TRY AGAIN" ELSE 5340
5320 PRINT:PRINT
5330 GOTO 5290
5340 INPUT "CORRECT VALUE";BF%(ROW%,COL%)
5350 CLS
5360 GOSUB 5960
5370 INPUT "CHANGE ANOTHER INPUT? (ENTER Y OR N)";ANSWER$
5380 IF ANSWER$="Y" OR ANSWER$="y" THEN 5290
5390 GOSUB 6090:IF BF%(1,1)>BF%(2,1) THEN 5770
5400 IF BF%(1,1)=BF%(2,1) AND BF%(1,2)>BF%(2,2) THEN 5770
5410 IF BF%(1,1)=BF%(2,1) AND BF%(1,2)=BF%(2,2) AND
     BF%(1,3)>=BF%(2,3) THEN 5770
5420 IF BF%(1,1)>MO% OR BF%(2,1)>MO% THEN 5770
5430 IF BF%(1,1)=MO% AND BF%(1,2)>DAY% THEN 5770
5440 IF BF%(2,1)=MO% AND BF%(2,2)>DAY% THEN 5770
5450 IF BF%(1,1)=MO% AND BF%(1,2)=DAY% AND BF%(1,3)>TBASE
     THEN 5770
5460 IF BF%(2,1)=MO% AND BF%(2,2)=DAY% AND BF%(2,3)>TBASE
     THEN 5770
5470 TDIF1 =FNTDIF1(BF%(2,1), BF%(2,2), BF%(2,3), BF%(1,1),
     BF%(1,2), BF%(1,3))
5480 IF BF%(2,4)=0 THEN BFR=0 ELSE 5510
5490 LPRINT "BFR=";BFR, "BASEFLOW =0"
5500 GOTO 5810
5510 BFR=(BF%(1,4)/BF%(2,4))^(1/TDIF1)
5520 LPRINT "TIME DIFF. =";TDIF1
5521 LPRINT "FNTIME(H2) =";FNTIME(BF%(2,3)),"FNTIME(H1)
     =";FNTIME(BF%(1,3))
5530 IF TDIF1 <0 THEN 5770
5540 LPRINT "BFR =",BFR
5550 IF BFR <= 1.04 THEN 5580
5560 LPRINT "BASEFLOW RECESSION HIGH.....ENTER PRECIP.
     DATA FOR PREVIOUS STORM
5570 GOTO 3120
5580 TDIF2 =FNTDIF1(MO%, DAY%, TBASE, BF%(2,1), BF%(2,2),
     BF%(2,3))
5590 LPRINT "TIME FROM LAST BF DATA POINT TO TBASE =";TDIF2
5600 IF TDIF2 < 0 THEN 5770
5610 FOR J=1 TO 240
5620 FLOSUM%(15,J)=BF%(2,4)/(BFR^(TDIF2+(J*.25)))
5630 NEXT J
5640 GOTO 5810
```

```
5650 TDIF3= FNTDIF1(BF%(2,1), BF%(2,2), BF%(2,3), BF%(3,1),
     BF%(3,2), BF%(3,3))
5660 LPRINT "TIME 1ST PEAK TO 2ND RECESSION POINT =";TDIF3
5670 F16=BF%(2,4)/BFR^(16-TDIF3)
5680 TDEL=16-TDIF3
5690 FOR J1=1 TO 240
5700 TBF=TDIF2 + (J1*.25)
5710 IF TBF<= TDEL THEN 5740
5720 FLOSUM%(15,J1)=F16/(1.005^(TBF-TDEL))
5730 GOTO 5750
5740 FLOSUM%(15,J1)=BF%(2,4)/(BFR^TBF)
5750 NEXT J1
5760 GOTO 5810
5770 CLS
5790 LPRINT:LPRINT
5800 GOTO 5360
5810 RETURN
5820 REM ***SUBROUTINE TO DISPLAY BASEFLOW INPUT HEADER ***
5830 PRINT TAB(35)"COL#1" TAB(45)"COL#2" TAB(55)"COL#3"
     TAB(65)"COL#4"
5840 PRINT
5850 PRINT TAB(35)"MONTH" TAB(45)"DAY" TAB(55)"TIME"
     TAB(65)"FLOW"
5860 PRINT TAB(35)"NUMBER" TAB(45)"NUMBER" TAB(55)"HRMN"
     TAB(65)"CFS"
5870 PRINT
5880 RETURN
5890 REM ***SUBROUTINE TO PRINT BASEFLOW INPUT HEADER ******
5900 LPRINT TAB(35)"COL#1" TAB(45)"COL#2" TAB(55)"COL#3"
     TAB(65)"COL#4"
5910 LPRINT
5920 LPRINT TAB(35)"MONTH" TAB(45)"DAY" TAB(55)"TIME"
     TAB(65)"FLOW"
5930 LPRINT TAB(35)"NUMBER" TAB(45)"NUMBER" TAB(55)"HRMN"
     TAB(65)"CFS"
5940 LPRINT
5950 RETURN
5960 REM ***SUBROUTINE TO DISPLAY BASEFLOW INPUT DATA ******
5970 CLS
5980 IF BF%(1,4)>BF%(2,4) THEN 6020
5990 FOR J1=1 TO 4
6000 SWAP BF%(1,J1),BF%(2,J1)
6010 NEXT J1
6020 PRINT TAB(35)"   SUMMARY OF BASEFLOW INPUT DATA      "
6030 PRINT
6040 GOSUB 5820
6050 PRINT "ROW 1"; TAB(35) BF%(1,1); TAB(45) BF%(1,2);
     TAB(55) BF%(1,3); TAB(65)BF%(1,4)
6060 PRINT "ROW 2";TAB(35) BF%(2,1);TAB(45) BF%(2,2);TAB(55)
     BF%(2,3);TAB(65)BF%(2,4)
```

```
6070 PRINT
6080 RETURN
6090 REM ****SUBROUTINE TO PRINT BASEFLOW INPUT DATA ******
6100 IF BF%(1,4)>BF%(2,4) THEN 6140
6110 FOR J1=1 TO 4
6120 SWAP BF%(1,J1),BF%(2,J1)
6130 NEXT J1
6140 LPRINT TAB(35)"   SUMMARY OF BASEFLOW INPUT DATA      "
6150 LPRINT
6160 GOSUB 5890
6170 LPRINT "ROW 1";TAB(35) BF%(1,1);TAB(45) BF%(1,2);TAB(55)
     BF%(1,3);TAB(65)BF%(1,4)
6180 LPRINT "ROW 2";TAB(35) BF%(2,1);TAB(45) BF%(2,2);TAB(55)
     BF%(2,3);TAB(65)BF%(2,4)
6190 LPRINT
6200 RETURN
6210 REM******* SUBROUTINE TO ENTER PRECIP DATA *************
6220 CLS
6230 PRINT TAB(20) "ENTER PRECIP DATA FOR STORM #";ST%
6240 PRINT:PRINT
6250 PRINT TAB(25)"COL#1" TAB(35)"COL#2" TAB(45)"COL#3"
     TAB(55)"COL#4" TAB(65)"COL#5"TAB(75)"COL#6"
6260 PRINT:PRINT
6270 PRINT TAB(25)"STORM" TAB(35)"5-DAY" TAB(45)"START"
     TAB(55)"DURATION"
6280 PRINT TAB(10)"SUBBASIN" TAB(25)"PRECIP" TAB(35)"PRECIP"
     TAB(45)"TIME"TAB(55)"(HOURS)"TAB(65)"MONTH"TAB(75)"DAY
6290 PRINT :PRINT
6300 FOR KK1 = 1 TO 13
6310 PRINT "ROW# ";KK1,NNAME$(KK1);TAB(25)
6320 INPUT; DAT(KK1,1,ST%)
6330 IF DAT(KK1,1,ST%)=0 THEN DAT (KK1,1,ST%) =
     DAT(KK1-1,1,ST%)
6340 PRINT TAB(35)
6350 INPUT; DAT(KK1,2,ST%)
6360 IF DAT(KK1,2,ST%)=0 THEN DAT (KK1,2,ST%) =
     DAT(KK1-1,2,ST%)
6370 PRINT TAB(45)
6380 INPUT; DAT(KK1,3,ST%)
6390 IF DAT(KK1,3,ST%)=0 THEN DAT(KK1,3,ST%)=DAT(KK1-1,3,ST%)
6400 PRINT TAB(55)
6410 INPUT;DAT(KK1,4,ST%)
6420 PRINT TAB(65)
6430 IF DAT(KK1,4,ST%)=0 THEN DAT (KK1,4,ST%) =
     DAT(KK1-1,4,ST%)
6440 INPUT;DAT(KK1,9,ST%)
6450 IF DAT(KK1,9,ST%)=0 THEN DAT(KK1,9,ST%)=DAT(KK1-1,9,ST%)
6460 PRINT TAB(75)
6470 INPUT DAT(KK1,10,ST%)
6480 IF DAT(KK1,10,ST%)=0 THEN DAT(KK1,10,ST%) =
```

```
     DAT(KK1-1,10,ST%)
6490 NEXT KK1
6500 GOSUB 3140
6510 INPUT "ARE INPUT CORRECTIONS NEEDED? (ENTER Y OR
     N)";ANSWER$
6520 PRINT
6530 IF ANSWER$ = "N" OR ANSWER$="n" THEN 6640
6540 INPUT;"ROW# ";ROW%
6550 INPUT;"     COLUMN# ";COL%
6560 IF COL% = 5 THEN COL% = 9
6570 IF COL%=6 THEN COL%=10
6580 INPUT "     CORRECT VALUE";DAT(ROW%,COL%,ST%)
6590 GOSUB 3140
6600 PRINT
6610 INPUT "CHANGE ANOTHER INPUT? (ENTER Y OR N)";ANSWER$
6620 IF ANSWER$ ="Y" OR ANSWER$="y" THEN 6540
6630 CLS
6640 GOSUB 3350:RETURN
6650 REM**** SUBROUTINE TO PRINT BASIN HYDROGRAPH ****
6660 TOUT=FNTIME(TBASE)
6670 DOUT%=DAY%
6680 MOUT%=MO%
6690 T1OUT=TOUT+20
6700 T2OUT=TOUT+40
6710 D1OUT%=DOUT%
6720 D2OUT%=DOUT%+1
6730 MOUT%=MO%
6740 M1OUT%=MOUT%
6750 M2OUT%=MOUT%
6760 IF T1OUT>=24 THEN T1OUT=T1OUT-24 : D1OUT%=D1OUT%+1
6770 IF T2OUT>=48 THEN T2OUT=T2OUT-48 : D2OUT%=D2OUT%+1
6780 IF T2OUT>=24 THEN T2OUT=T2OUT-24
6790 IF D1OUT%>DAYS(MOUT%) THEN D1OUT%=D1OUT%-DAYS(MOUT%):
     M1OUT%=M1OUT%+1
6800 IF D2OUT%>DAYS(MOUT%) THEN D2OUT%=D2OUT%-DAYS(MOUT%):
     M2OUT%=M2OUT%+1
6810 LPRINT " DATE    HOUR  FLOW     DATE    HOUR  FLOW
     DATE    HOUR  FLOW"
6820 FOR I=1 TO 80
6830 TOUT=TOUT+.25
6840 T1OUT=T1OUT+.25
6850 T2OUT=T2OUT+.25
6860 IF TOUT>=24 THEN TOUT=TOUT-24: DOUT%=DOUT%+1
6870 IF T1OUT>=24 THEN T1OUT=T1OUT-24 : D1OUT%=D1OUT%+1
6880 IF T2OUT>=24 THEN T2OUT=T2OUT-24 : D2OUT%=D2OUT%+1
6890 IF DOUT%>DAYS(MOUT%) THEN DOUT%=1 : MOUT%=MOUT%+1
6900 IF D1OUT%> DAYS(MOUT%) THEN D1OUT%=1 : M1OUT%=M1OUT%+1
6910 IF D2OUT%> DAYS(MOUT%) THEN D2OUT%=1 : M2OUT%=M2OUT%+1
6920 LPRINT USING"##/## #### #####     ";MOUT%, DOUT%,
     FNT24(TOUT), FLOSUM%(14,I)*2, M1OUT%, D1OUT%,
```

```
     FNT24(T1OUT), FLOSUM%(14,I+80)*2, M2OUT%, D2OUT%,
     FNT24(T2OUT), FLOSUM%(14,I+160)*2
6930 NEXT I
6940 RETURN
6950 REM***SUBROUTINE TO IDENTIFY AND PRINT PEAK FLOW & TIME
     OF PEAK******
6960 FOR I = 1 TO 240
6970 IF FLOSUM%(14,I) >= QMAX%(14) THEN QMAX%(14) =
     FLOSUM%(14,I): PKOUT%=I
6980 NEXT I
6990 TPKOUT = FNTIME(TBASE) + PKOUT%/4
7000 DOUT% = DAY%
7010 MOUT% = MO%
7020 IF TPKOUT>=72 THEN TPKOUT=TPKOUT-72:DOUT%=DOUT%+3:
     GOTO 7050
7030 IF TPKOUT>=48 THEN TPKOUT=TPKOUT-48:DOUT%=DOUT%+2:
     GOTO 7050
7040 IF TPKOUT>=24 THEN TPKOUT=TPKOUT-24:DOUT%=DOUT%+1:
     GOTO 7050
7050 IF DOUT%>DAYS(MOUT%) THEN DOUT% =
     DOUT%-DAYS(MOUT%):MOUT% = MOUT%+1
7055 DUM=QMAX%(14): DUM=DUM*2
7060 LPRINT USING"PEAK FLOW = ##### CFS  ON ##/##  @
     ####";DUM,MOUT%,DOUT%,FNT24(TPKOUT)
7070 RETURN
7080 REM*****SUBROUTINE TO MODIFY BASEFLOW FOLLOWING PEAK****
7090 FLO=FLOSUM%(KK7,MARK%(KK7))
7100 FOR K9%=MARK%(KK7)+1 TO 240
7110 FLO=FLO/1.005
7120 FLOSUM%(14,K9%)=FLOSUM%(14,K9%)-(FLOSUM%(KK7,K9%)/2)
7130 FLOSUM%(KK7,K9%)=FLO
7140 FLOSUM%(14,K9%)=FLOSUM%(14,K9%)+FLOSUM%(KK7,K9%)/2
7150 NEXT K9%
7160 RETURN
7170 REM****SUBROUTINE TO STORE FLOW MATRIX AND ADD TRAILING
     BASEFLOW
7180 OPEN "C:\HYDRO.DAT" FOR OUTPUT AS #1
7200 PRINT#1,ST%,MO%,DAY%,TBASE
7210 FOR I= 1 TO 13: PRINT#1, MARK%(I): NEXT I
7220 FOR I= 1 TO 14: PRINT#1, QMAX%(I): NEXT I
7230 FOR I= 1 TO 13: PRINT#1, CUMP(I): NEXT I
7240 FOR I= 1 TO 13: PRINT#1, CUMSRO(I): NEXT I
7250 FOR I= 1 TO 14
7260 FOR J= 1 TO 240: PRINT#1,FLOSUM%(I,J):NEXT J
7270 NEXT I
7280 FOR KK7=1 TO 13: GOSUB 7080:NEXT KK7
7310 CLOSE
7320 IF OUTP%=1 THEN GOSUB 6950 ELSE GOSUB 6650
7330 RETURN
7340 REM****SUBROUTINE TO RETRIEVE PREVIOUS STORM
```

```
     HYDROGRAPHS*****
7350 OPEN "C:\HYDRO.DAT" FOR INPUT AS #1
7360 INPUT#1,ST%,MO%,DAY%,TBASE
7370 FOR I= 1 TO 13: INPUT#1, MARK%(I): NEXT I
7380 FOR I= 1 TO 14: INPUT#1, QMAX%(I): NEXT I
7390 FOR I=1 TO 13: INPUT#1, CUMP(I): NEXT I
7400 FOR I=1 TO 13: INPUT#1, CUMSRO(I): NEXT I
7410 LPRINT "PREVIOUS # STORMS = ";ST%,"     TIMEBASE =
     ";MO%;"/";DAY%;"/";TBASE
7420 ST%=ST%+1
7430 FOR I = 1 TO 14
7440 FOR J = 1 TO 240 : INPUT#1, FLOSUM%(I,J): NEXT J
7450 NEXT I
7460 CLOSE
7470 RETURN
```

Summary documentation for key variables.

BF% - Baseflow data matrix containing month, day, hour, and discharge (cfs) for two points on hydrograph prior to storm.

BFR - Baseflow recession constant (dimensionless) calculated from baseflow data prior to storm.

CN1,2,or3 - SCS curve number for subbasins under AMC I, II, or III conditions (read from DATA statements for each subbasin).

CUMP - Matrix of accumulated subbasin precipitation (inches).

CUMSRO - Matrix of accumulated subbasin surface runoff (inches).

DAT - Matrix of storm input data and selected output: rainfall (inches); five-day antecedent rainfall (inches); time that storm began (month,day,and hour); storm duration (hours); computed SCS curve number; computed runoff (inches); computed peak subbasin discharge at Ames (cfs), time of subbasin peak flow in Ames (hours).

DAYS - Matrix containing number of calendar days in each month.

FLOSUM%	-	Matrix of predicted discharge (15-minute increments), at Ames, for each subbasin. Last row contains sum of flows from each subbasin.
NNAME$	-	Matrix of subbasin names.
NRISE%	-	Percent of peak flow (15-minute increments) for each subbasin (these are read from DATA statements).
QMAX%	-	Matrix of peak discharge, at Ames, for each subbasin.
QPK	-	Subbasin peak discharge (at Ames) for an individual storm.
QSLOPE	-	Slope factor (cfs/inch of runoff) for a subbasin (read from DATA statement for each subbasin).
TBASE	-	Clock time (24-hour clock) when rainfall began in the basin.
TPK1	-	Time (hours) between rainfall initiation in a subbasin and arrival of peak discharge (at Ames) for a one-hour duration storm (read from DATA statement for each subbasin).