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Weather radar-based runoff modeling for the Squaw Creek watershed in central Iowa

by

Murielle Marie-Laure Maud Jeanne

A thesis submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of
MASTER OF SCIENCE

Major: Civil Engineering (Environmental Engineering)
Major Professors: T. Al. Austin and Larry L. Northup

Iowa State University
Ames, Iowa
1998

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Graduate College
Iowa State University

This is to certify that the Master's thesis of

Murielle Marie-Laure Maud Jeanne

has met the requirements of Iowa State University

Signatures have been redacted for privacy

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To Jason, my husband, the most wonderful person I know;

To Keith and Karen, my fantastic adoptive parents who care so much for me;

To June, my dearest adoptive grandmother whose mind is my twin;

To Mamie Louise, my delightful grandmother, whose still lives in my memory, and to my caring grandfather Papy Edouard.

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"Floods have been many things to many people. To Noah and his people the Deluge was a manifestation of a wrathful God. To the Pharaohs 16 'ells' on the Nile gage meant *wafa* -a period of abundance, a contented people, and above all a freedom to tax without fear of unrest. To the people of the Far East floods have made plains and deltas on which to subsist in spite of a lifelong threat of death by drowning. In the United States they probably stimulate more interest among more diverse groups than any other natural phenomenon.

To reporters and editors of the daily press they are dramatic news.

To those caught in the swirling mud-ladden waters they represent days and nights of terror.

To the Red Cross workers, National Guard units, or citizens huriedly pressed into service to carry sandbags or to contribute money, food, and clothing, or to provide shelter for the homeless, they represent a call to action, a summons that is gladly met.

To hydrologists floods mean the immediate translation of measurements of rain, snow, wind, and ground conditions into forecasts of river stages and their widespread dissemination by radio, telephone, and the press. If the danger is overestimated, the forecasting service may be discredited; if it is underestimated, the result is added damage and loss of life.

To river engineers floods mean measurements of stages and volumes under the worst possible conditions; the study of ways to reduce damage; and the design, coonstruction, and operation of extensive dikes, levees, flood walls, channels, dams, and reservoirs -protective works which must not fail but which must function effectively to the limits of their designed capacity.

To farmers they mean erosion and loss of topsoil on barren or newly planted side-hill fields, and inundation and flooding of crops on valley bottom lands.

To conservationists the gullied fields and loss of valuable topsoil indicte a need for improved land-use practices and measures to increase infiltration and retard erosion.

To legislators and constitutional lawyers floods have often represented a peg on which to hang measures an d practices apparently designed not so much to reduce flood damage as to stabilize the economy, promote the general welfare, or in some instances just curry political favor [...].

To water users, especially in the West, they mean recharged ground water and the filling of storage reservoirs.

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To city and state planning boards and commissions they represent a need for rules and regulations whereby flood-hazard lands may be appropriately utilized.

To statisticians they mean a series of events among which they look in vain for the alchemy of cyclic variations.

To insurance executives they represent almost the only loss or damage that apparently cannot be profitably underwritten.

To bankers they reflect the size and suitability of loans.

To political scientists and economists they may present a picture of management or mismanagement which can only be corrected by more federal control, more regional authorities, or more state and local participation, depending on the point of view.

To some people floods may be only a source of inconvenience; to others they may be an immediate danger; to all of us as citizens they are a natural phenomenon on which we will have spent [several] million dollars under our present flood-control policy."

(Reprinted from Hoyt and Langbein, 1955)

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ACKNOWLEDGEMENTS

I especially want to thank Troy Nicolini, formerly Hydraulic Engineer (Research Division) at the Hydrologic Engineering Center (HEC) in Davis, CA, and now Service Hydrologist at the National Weather Service in Eureka, CA, for his outstanding support, guidance, and answers to my countless questions during our collaboration work. I greatly appreciated his trustand encouragement, and am very thankful for his friendship.

I would like to thank Tom Evans, Hydraulic Engineer at HEC (Technical Assistance Division) for his help and the time spent to answer my GIS questions.

I am very grateful to Carl Franke, Hydraulic Engineer at HEC (Technical Assistance Division), who took the time to help with the NEXRAD rainfall data obtainment and processing problems.

I give my thanks to Bill Scharffenberg, Hydraulic Engineer (Research Division) at HEC, for having provided insight into the problems encountered with the radar data computations.

I also thank Al Montalvo, Senior Hydraulic Engineer at HEC (HEC (Technical Assistance Division), who provided technical computer aid on GSS problems.

I am thankful to Karl Jungbluth, Science and Operations Officer at the National Weather Service in Johnston, IA, who contributed to the rainfall events' selection, helped with different details concerning the rainfall data, and provided images of the Stage I data for the events.

I appreciate the cooperation of Jayne May, USGS Supervisory Hydrologist in Iowa City, IA, and the efforts of Alvin Conkling, USGS Lead Hydrologic Technician in Fort Dodge, IA, and Von Miller, USGS Lead Hydrologic Technician in Iowa City, IA, who prepared and provided the necessary hourly streamflow data for the project.

I wish to express my great appreciation to Dr. Austin and Dr. Northup from Iowa State University Department of Environmental Engineering for their great trust, advice, and support during the whole project.

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Last, but not least, my special thanks go to my husband, Jason, for his love, patience, and support during my work, especially during the times of greater frustration.

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CHAPTER 1. INTRODUCTION

"I have never seen anything on this scale before... It was awful", proclaimed President Bill Clinton, touring Des Moines by helicopter on an inspection visit to Iowa in early July 1993 (Cochran, 1993). The floods of the 1993 summer were indeed terrible and the costliest and most devastating floods in US history since 1930 (Cochran, 1993; Mairson, 1994; National Weather Service, 1994). Nine states -North Dakota, South Dakota, Minnesota, Wisconsin, Iowa, Nebraska, Illinois, Missouri and Kansas-, more than 15% of the contiguous US, were catastrophically impacted. A wet Fall, premature snowmelt in the Spring, and heavy rains in the Summer resulted in above-normal soil moisture and water storage conditions in the Upper Mississipi and Missouri River basins. Record breaking floods on the Missouri and the Mississipi rivers rampaged through the Midwest. The summer saw heavy, unprecedented storms and unrelenting rainfall on Minnesota, Nebraska, Iowa, Kansas, Illinois and Missouri. During June and July, heavy rains fell 39 out of 54 days. The Missouri and Mississipi, destroying and washing away everything in their paths, crested at all time highs. Through the middle of August, Iowa had 12 consecutive weeks of above-normal rainfall. The average state rainfall for July was more than 25 centimeters, the highest July total in 121 years of record keeping. In Ames, central Iowa, both the Skunk River and Squaw Creek rose above their banks. On the Iowa State University campus, Hilton Coliseum filled with over 4 meters of water-up to the first row of parquet-level seats. University buildings sustained an estimated 7.7 millions of dollars in damage. The whole city of Ames had over 10 million dollars total flood damages (Tebben et al., 1997). By the time the rivers quit raging in Iowa, seven people had died; more than 21,000 houses, apartments and mobile homes were destroyed or damaged; crop losses were estimated to be at least 1 billion dollars and rising as farmers stared at an uncertain harvest, and total damage was conservatively estimated at 27 billion. "We just hope this is the last one", said Jerry Veit of Chelsea, Iowa (Cochran, 1993).

Unfortunately, no one could assure him that such an extreme flood will not occur again. Why? Because floods are natural events. As emphasized by Dr.

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G. Galloway (1997, personal communication), no one across the world knows why floods occur. They are normally occurring events and, because of this inherent nature, are simply inevitable. Damages caused by flooding are however not natural events. Focusing on trying to reduce the devastating consequences of flooding represents our only asset in this battle against extreme natural events. It is estimated that about 17,000 cities in the US have flood prone areas. Residential and business development have always historically taken place in floodplain areas since rivers are a key to development (Cooper, 1993). Throughout history, people have always settled next to waterways because of the advantages they offer in transportation, commerce, energy, water supply, soil fertility and waste disposal (Montgomery, 1989). The source of wealth and way of life along the river formed the main vicinity for work places, which forced towns to settle in floodplains. A better management and regulation of use of floodplains by human activities could possibly mitigate the impacts of a flood on people's lives and properties, but is not easy to devise due to the intricacies of economical and political considerations that come into play. Preventing development in hazardous land tracts would be one way to regulate land use in the river basin. The relocation of endangered buildings or buildings that have already been flooded several times could help diminish the flood threat. Finally, the development of performant early flood warning systems is necessary in areas that have been populated for a long time and where the potential for loss of life and property damage is real.

Accurate and rapid forecasting of high flow events in floodplain areas constitutes a significant challenge for planners and city officials. For the last few decades, the flood phenomenon has been the object of particular attention from scientists as well as from government, because of the human and economic damage it could generate. Computers have revolutionized the flood warning technology. Sophisticated systems of flood forecasting have been developed around the world (see Chapter II). They combine remote rain gauges and river stage instruments with powerful software run on base station computers. The traditional approach to flood forecasting is to use rainfall input estimated from a number of rain gauges, driving a lumped (spatially averaged) parameter hydrological model. The advent of new technologies such as weather radars and satellites, Geographic Information

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Systems (GISs), and high-speed computer workstations, provides new opportunities for improved hydrologic forecasting. The imput that drives most hydrologic models is precipitation, which has never been widely available on a spatially distributed basis. It is true that, with a rain gauge network of about 1 raingage per 52 to 520 km^2, the available data in some areas of the US are sufficient so that some level of rainfall-runoff modeling using distributed parameter sets and distributed precipitation inputs could be accomplished to some extent. Hydrologists responsible for operational forecasting have used various techniques to compute inputs from existing precipitation gauge networks used to drive rainfall-runoff models.

Currently, NEXRAD (Next Generation Weather Radar) WSR-88D (Weather Surveillance Radar-1988 Doppler) radars have been installed at various locations across the US (see Appendix B) to provide radar coverage over essentially the entire country. Information from the WSR-88D can be processed and quality controlled through a series of precipitation data processing programs, resulting in quantitative precipitation estimates at a 4 km grid scale and 1 hour time intervals. The high spatial and temporal resolution of these precipitation data provides hydrologists with new input to rainfall-runoff models. Such input has never before been available. The WSR-88D represents a significant milestone for hydrologic modeling because it will allow for gridded, real-time and quantitative estimates of precipitation to be processed. The availability of NEXRAD rainfall data enhances the use of distributed simulation approaches that take into account spatial variations of rainfall.

The Squaw Creek basin in central Iowa has generated several major floods over the past century. Ames, a city of 48,000 inhabitants, is located within the watershed. Although several studies have been undertaken by different agencies and a city flood warning system designed, the potential for a damaging flash flood event remains (Glanville, 1987; Tebben, 1997). Devising an optimum flood warning system is the best solution to such a threat to human life and property. The use of a system coupling WSR-88D rainfall data with a runoff hydrologic model could represent an adequate method of flood forecasting in the Squaw Creek river basin and would be an asset to the current Ames flood warning system. Collaborative work has been initiated in 1996 between the Iowa State University Civil Engineering Department and the

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Hydrologic Engineering Center (HEC) in Davis, CA. The HEC research group has recently devised a new modeling approach called the ModClark River Basin Analysis Procedure, that integrates NEXRAD rainfall data and GIS Digital Elevation Models (DEMs) into hydrologic modeling. This new rainfall-runoff modeling methodology is included in HEC's Hydrologic Modeling System, HEC-HMS. It is part of the HEC NexGen Software Project, which is developing successor generation sofware packages for use in hydrologic engineering, water resource planning and project operation. The ModClark procedure is new and needs some testing for its application to flat area basins. The topographical configuration of the Squaw Creek watershed, typical of Iowa plains, made it an interesting site for testing the model. The ModClark model was developed for the Squaw Creek basin. The model was then verified and adjusted with some real historical storm event simulations.

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CHAPTER 2. LITERATURE REVIEW

"Those who cannot remember the past are condemned to repeat it", said George Santayana in 1905 (Keller and Capelli, 1992). It is a common place that those who always ignore lessons of the past must bear the consequences when things happen again. The repetitive nature of many natural phenomena such as floods is undisputed (Keller and Capelli, 1992). Floods are dangerous, life-threatening and destructive (Boning and Stallings, 1992). Accounts of floods and flood disasters appear in numerous biblical writings (Hoyt and Langbein, 1955). The Romans used dams and diversions in attempting to reduce potential floods. Floods have been the scourge of India, China and other countries of the Far East. In Europe, Italy, France and Germany in particular have a history of record floods. Concerns about floods have existed in the United States since the first settlers arrived. Floods are the most common and widespread of all natural disasters -except fire- and occur within all 50 states (FEMA, 1996). They have caused a greater loss of life -death of more than 10,000 people since 1900- and property -total over $1 billion each year-, and have disrupted more families and communities in the US than all other natural hazards combined. The earth has a history of repetitive events so it can be argued that a fair understanding of the historical behavior of a river can enhance environmental planning and reduce all kinds of disasters stemming from natural physical processes. But things are not as straightforward in reality. As from ancient times, human settlements have usually been located close to a river (Cooper, 1993; Montgomery, 1989). Old cities were situated on river banks because the river provided water supply, transportation, and took away any waste. The fine sediment deposited over the floodplain during flooding made soils especially fertile. In addition to providing scenic features to live near, the stream also represented a natural defense mechanism. These places have evolved in time but their location has remained unchanged. Nowadays, development in these floodplain areas has become intensive and keeps on expanding. Flooding events still occur, having a potentially greater impact on riparian communities as urbanization intensifies and natural flood control systems such as native vegetation cover give way to impervious

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surfaces. Efforts to reduce flood-related deaths and damages have therefore been increasing. Many different measures for coping with floods have been developed over the course of time (Yevjevich, 1994). The most well-known structural measures are levees, dams and storage reservoirs, towards which efforts were primarily directed. Non-structural measures consist mainly of floodplain regulation, insurance and flood forecast and represent "newer" approaches (Boning and Stallings, 1992). Interest in flood forecasting has particularly increased in the last 20 years. The need for protection against floods is critical for communities located close to a river. Having an adequate early warning system that can be relied upon therefore represents an extremely useful asset.

2.1. History of flood warning

Early flood-warning systems for people living along streams involved personal travel and verbal exchange of information (Boning and Stallings, 1992). Communication systems such as the telephone greatly improved the timeliness of flood warnings. Federal, state and local water management agencies began to remotely access data from the US Geological Survey (USGS) gaging stations in the 1930's by an instrument called "Telemark". When accessed by telephone, the Telemark transmitted river flow stage by a series of beeps or rings. High frequency radio also began to be used in the 1930's to obtain rivers stage data. By the 1950's, river stage at hundreds of USGS gages throughout the US could be accessed to forecast floods, provide flood warnings, or help water management for drought situations. The continued evolution of communication and stage-sensing equipment has further improved data access for flood-warning and flood-forecasting purposes (Chow et al., 1988; Clark, 1994; Latkovich and Leavesley, 1993; National Weather Service, 1994; Swain, 1996; US Geological Survey, 1996; Wennenberg, 1985). The principal devices currently in use in USGS gages for obtaining near real-time data for flood warning or other water-mangement purposes are those that transmit data via satellite to a receiving station where the data are then relayed with conventional radio and telephone systems. Microwave and satellite transmission of data are valuable for contributing to the timeliness of data critical to flood forecasting. They also provide continuous access to remote recording sites which are difficult to access by land.

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The initial use of remotely accessed data to forecast floods was often limited to the interpretation of correlative relations that existed between different streams or stream locations (Boning and Stallings, 1992). This required a great deal of intuitive expertise and knowledge on the part of the river or weather forecaster. The rapid development of more sophisticated computer systems since the 1970's, however, has permitted large amounts of data to be incorporated into computer-simulation models (Nelson, 1992). Computers have revolutionized flood warning technology by incorporating remote rain gage and river stage data with powerful software, which is of critical use to provide warnings or forecasts of floods.

2.2. River forecasting at the National Weather Service

The National Weather Service (MWS), which is part of the National Oceanic and Atmospheric Adminstration (NOAA) is the federal agency in charge of weather forecasting and warning for the nation (Burnash, 1995; Mason and Weiger, 1995). The mission of the NWS's Hydrologic Services is to save lives and decrease property damage by issuing timely flood warnings and river stage forecasts. Although many cities, counties or other local flood-management agencies are involved in the operation of local flood-warning networks, the NWS, through its nationwide hydrologic-forecasting mandate, is the principal agency that uses non structural methods to decrease flood damage (Boning and Stallings, 1992). Flooding along major rivers that is caused by rainfall takes many hours and even weeks to develop. Floods caused by snowmelt runoff may take up to several months to develop. Flash floods occur when intense precipitation falls during a brief time span on smaller rivers. The time between the onset of this intense precipitation and the cresting of the river is hours instead of days. More than 10,000 precipitation and streamflow stations provide hydrologic data to NWS offices across the country for use in flood forecasting. The USGS is the principal source of data on river flow depth and discharge (Mason and Weiger, 1995). Hydrologic data collection at the stream gaging station is telemetered through the Geostationary Operational Environmental Satellite (GOES) to regional NWS River Forecast Centers (RFCs). Across the US, 13 RFCs monitor the nation's river system and are responsible for flood warnings within at least one major river system (National Weather Service Communications Group, 1996b). To

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develop flood forecasts, the RFCs develop and calibrate complex mathematical models of how the rivers respond to snowmelt and rainfall (Page and Smith, 1994; National Weather Service, 1996a). The technology of the Sacramento Catchment Model is the primary modeling component, with spatially lumped parameters (Burnash, 1995). Rainfall data are entered into the model, which estimates the river stage and discharge that will result. This is used in making hydrologic forecasts and advisories (Mason and Weiger, 1995; National Weather Service, 1994). These models are developed for preselected forecast service points --about 4,000 across the nation --, which are usually located along major rivers or near urban areas having a history of flooding.

2.3. The arrival of NEXRAD radars

Expanded use of telemetry at the USGS streamflow stations and refinement of telemetry equipment continue to improve the timeliness and reliability of data that are transmitted for forecasting purposes (Boning and Stallings, 1992). Nevertheless, a new era of river and flood forecasting is beginning (Shedd and Fulton, 1993). The NWS is beginning a major modernization of its forecasting system and hydrologic forecast operations in the future will differ dramatically from those in the pre-modified NWS. The goal of this modernization effort is to create an integrated forecast environment for improved and effective forecasting (Fread et al., 1991; Hudlow, 1988; Shelton and May, 1996). In addition to a networked suite of scientific superspeed workstations and tools appropriate to specific weather problems, the Advanced Weather Interactive Processing System (AWIPS) will include the implementation of NEXRAD into the hydrometeorological forecast applications defined in the modernized NWS. AWIPS is a high-speed weather computer and communication network that enables forecasters to collect and manage complicated meteorological information promptly and then interpret it within a few seconds (Reed, 1997). AWIPS national network is scheduled to be deployed mid 1999. The end goal of the modernization effort is to develop the capability to issue warnings of flood and flash flood events in real-time. Conventional NWS WSR-57 and WSR-74 radars have been progressively replaced by the new WSR-88D radar system since 1991, which is the keystone of the system modernization.

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According to the NWS, the WSR-88D system represents a key component in warning improvement (Fulton, 1997; Polger et al., 1994). The advantages of the WSR-88D over conventional radars can be broken into four basic areas: improved sensibility, improved resolution, automatic volume scanning and enhanced capabilities (see Appendix B). The performance of the severe local storm and flash flood warning programs at five NWS offices -in Florida, Virginia, Missouri, Kansas and Texas- before and after the availability of the WSR-88D was quantitatively investigated in 1993 (Polger et al., 1994). Resulting statistics showed that the warnings improved dramatically when the WSR-88D was in operation. In particular, the probability of detection of severe weather events increased and the number of false alarms decreased. There was also a marked improvement in the lead time -time from the issuance of the warning to the beginning time of the event- for all severe local storm and flash flood events. In Arkansas in Spring 1997, forecasters using Doppler radar were able to issue timely and accurate warnings of tornadoes, giving residents as much as 32 minutes to seek shelter -compared with the usual national average warning time of 11 minutes (Reed, 1997). Without the Doppler radar, casualties would have been far worse. The attractiveness of NEXRAD data for flood forecasting is the reason the NWS's goal is to use them in hydrologic modeling at the RFCs (National Weather Service Communications Group, 1997). Gridded radar high resolution quantitative estimates of precipitation can be processed into a time series of basin average precipitation values for a time duration of generally 1 hour (Shelton and May, 1996; Page, 1996). This pre-processing uses the output of the Stage III radar data (see Appendix B). The data can then be input into the NWS River Forecast System (NWSRFS), which is a modular graphical and interactive system containing a variety of models and procedures (Hudlow, 1988; Page, 1996; Page and Smith, 1996). The NWS is currently in the midst of the modernization program and work is ongoing on the integration of NEXRAD radar data into the hydrologic forecast modeling system (Ingram et al., 1996; Page, 1996). Precipitation input to the NWSRFS has traditionally been through basin-average precipitation time series derived from available precipitation gage observations. With the arrival of spatially distributed rainfall estimates for the WSR-88D, changes need to be made to the NWSRFS to accommodate the new data by moving from lumped parameter modeling to more of a distributed parameter hydrologic modeling

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approach for river forecasting (National Weather Service, 1994; Smith et al., 1996b).

The use of NEXRAD rainfall data in flood forecasting is thus unprecedented and modeling with this type of precipitation data is just beginning. So far, the Arkansas-Red River Basin RFC has been the only one to test the new forecast program. The AWIPS has not been fully developed to all NWS offices yet (Page, 1996). Stage III radar data is also not available all over the nation at this point in time; the same is true for level II data (Cram, 1996; Cram, 1997) (see Appendix B). It is however clear that WSR-88D gridded quantitative rainfall estimates is of high interest for the improvement of forecasting capabilities. The fine spatial and temporal scales of the data are unique advantageous features promising a new future to river flow forecasting. Current literature reveals the existence of a general consensus about the subject. Diverse agencies, research centers and universities involved in hydrologic modeling foresee a great potential in taking advantage of the new technology (Birks et al., 1991; Borga and Di Luzio, 1992; Capovilla et al., 1991; DeVantier et al., 1993; Haggett et al., 1991; James et al., 1993; Mimikou et al., 1996; Ogden and Julien, 1994; Oliveira and Ford, 1991; Schultz, 1994). Recent advances in computer science and technology are now available to the practice of hydrology, which has become increasingly conmputational during the past recent years (Dodson, 1993; Engdahl and McKim, 1991). The widespread availability of digital computers with sufficient speed and storage capacity has rendered hydrologic computations easier. It is thus technically possible for these computer systems to supply all the hydrologic and meteorologic data required by hydrologic models. The integration and processing of spatially distributed radar rainfall data is technically completely feasible, though not simple.

2.4. Flood-forecasting methods currently used

This complexity of integration, added to the young age of the radar technology, might be responsible for the fact that several other methods of flood forecasting are currently still in use. In Japan, river forecasting of floods uses a runoff calculation method called Multi-Tank Matrix Method for Runoff (MTM method) (Okamoto, 1993). This computerized calculation, applied in an ungaged or gaged basin, determines the discharge-storage relation for a

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channel by both using hydrologic and hydraulic channel characteristics. A statistical approach to early flood warning is used in France by the Water Research Service of Electricité de France (EDF) (La Barbera et al., 1993; Obled, 1989). It is based on a collection of historical meteorologic files that are seasonally split. Almost all major events are currently detected up to 3 days in advance but on a very large space scale and with a high percentage of false alarms. French meteorologists however consider the statistical approach more reliable than meteorological models. The ARAMIS weather radar network of France is expanding to provide more accurate meteorologic data (Lanza et al., 1994). In Canada, one approach to basin flood forecasting is the use of a large amount of hydrologic -ie: surface soil moisture, surface soil temperature- and meteorological -ie: snow cover, radiation, precipitation- information online and available in real time (Whiting and Wheaton, 1987). It is fed into a hydro-meteorological system comprised of several computer modules analyzing the water budget, flow routing and hydrograph output used for flood warning. The processing speed of this system is of concern. In the Mediterranean Italian area, where flash floods rank high in the list of major hazards, early warnings rely on the accurate Quantitative Precipitation Forecasting (QPF) (La Barbera et al., 1993; Lanza et al., 1994). Meteorological data interpretation, validation, and integration is the key to hydrological forecasting and is based upon the joint use of remotely sensed information -mostly from METEOSAT- and ground measured data. The NOAA has developed a river forecast system for the Nile in Egypt (Barrett, 1993; Koren and Barrett, 1994). The system uses satellite imagery from the METEOSAT satellite as input. Precipitation data are then processed through a distributed water balance model and a routing model to produce a runoff forecast to predict inflows into the Aswan High Dam. A flood forecasting system based upon a raingage network is more commonly encountered (Billuart and Tourasse, 1980; Chow et al., 1988; Clark, 1994; Corradini et al., 1982; National Weather Service Communications Group, 1996b; Oliveira and Ford, 1991; Stanescu and Zanescu, 1980; Tebben, 1997; Wenyao et al., 1985). Gaged catchments' rainfall data obtained from the telemetered system has been traditionally used and input into rainfall-runoff models. A large number of rainfall-runoff models exist and are employed in various situations (Todini,

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1988; Peters, 1993). Examples include the ARNO model, HEC-1 and the Sacramento Watershed Model.

Satellite remote sensing data is also used in runoff models (Engman and Gurney, 1991). Reviewing literature shows that, in general, satellite imagery of various types is currently more commonly used than weather radar data in hydrologic modeling (Engman, 1993; Groves et al., 1985; Taillade, 1985; Tao and Kouwen, 1989; Schultz, 1994; Sharma et al., 1996).

2.5. Hydrologic modeling with radar data

Cases of research work using weather radar rainfall data for flood forecasting are not numerous. The Experimental Center for Hydrology and Meteorology in Teolo, Italy, is using weather radar rainfall estimates for high-flow forecasting (Capovilla et al., 1991). This center is the only one in Italy making use of such data for formulating forecasts. The radar used is a C-band Doppler dual-polarization radar operational since 1988. It is connected through a microwave link to the center as well as the telemetered network of meteorological stations. Research is currently ongoing so as to use the telemetric data from the ground network to assess the radar data measurements. A database management system allows the storage, retrieval and processing of data for input to the models. The forecast models used are of the lumped type: HEC1-F and ITFORMO. They process the mean areal rainfall computed for each subbasin from the precipitation data. In the UK, the use of C-band radar to provide quantitative rainfall information has been a growing feature of the Thames area's flood warning system these past years (Haggett et al, 1991). Both raingage network and single-site radar data are tele-transmitted to the control room for flood warning. Radar rainfall totals are calculated for various time periods and corrected, if judged necessary, with rainfall ground-based measurements using some empirically determined ratios of radar/gage data. Flood forecasting modeling is performed at some specific points -known to experience frequent flooding- by two different rainfall-runoff softwares: a synthetic unit hydrograph model to accommodate the flashy urban subcatchment around London, and a multiple-zone model for the rural larger catchments. Development and improvement of the flood forecasting system is being carried on. The Wessex region in the UK is also working on developing a similar integrated flow forecasting system

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(Birks et al., 1991). Research is also ongoing in the Wales region and the northwest part of the UK with the goal of establishing a real-time hydrologic forecasting model (Cluckie et al., 1987). Besides Italy and the UK, research work is carried on in Greece. A hydrometeorological study in a river basin has been conducted in central Greece to demonstrate the use of a weather radar for rainfall and flood flow forecasting (Mimikou and Baltas, 1994 and 1996). The HEC1-F rainfall-runoff model was applied twice, first by using as input the mean areal raingage rainfall derived by applying the Thiessen polygon method, and second by applying the mean areal radar (WSR-74 type) data over the basin. It was found that processed weather radar data merged with adequate raingage rainfall information can be effective for flood-flow forecasting. The rising limb and peak discharge of all streamflow hydrographs computed from weather radar data and calibrated with raingage data were more accurate than the corresponding ones from hydrographs computed from raingage rainfall data alone. Hudlow (1988) notes that other countries view weather radar as a principal system for improving rainfall observations and flood forecasting procedures. An example of work underway in Germany has been presented by Klatt and Schultz (1985). Their flood forecasting rainfall-runoff model called HYREUN relies upon the use of C-band radar rainfall data averaged over the whole basin and a rainfall forecasting procedure. No further information on their work can be found in current literature. The same is true for the WATFLOOD flood forecasting system, developed in Canada, which incorporates weather radar data in a 2 km square grid format (Kouwen, 1988). Data management programs include a simulation model called SIMPLE, which takes into account surface storage, infiltration and interflow, and an optimization algorithm for real-time operation. Japan is also moving towards the use of observations from weather radar (Lu, personal communication, 1997). A distributed hydrologic model, based on the Xinanjiang distributed model used in China (Zhao, 1992; Zhao and Liu, 1995), is currently under development (Lu et al., 1996a and 1996b). From a computed DEM and delineated channel network, hydrologic attributes are extracted from the 100 m square grid data. The computation of Horton-Strahler's stream order forms the basis for channel routing. The Japanese Ministry of Construction Yakushidake weather radar data grid --3 km square grid-- is then overlaid on the basin grid. The Xinanjiang model dates from

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1973 and is based on the concept of runoff formation on repletion of storage. The new model is not yet operational. All the countries cited are currently developing their weather radar networks, just beginning their research work, and published reports on their results are scarce.

It is interesting to note that, even though some of the above research cases use mean areal precipitation values from the radar data, results of flood forecasting are judged satisfactory. Both British and Greek authors indicate that weather radar's value in terms of the spatial and temporal variation of rainfall is beyond doubt. This is particularly apparent for local convective storms which can easily remain undetected by a conventional network of raingages and for frontal storms whose movement can be readily captured by the radar. It is also worth mentioning that all of the above cited research work applies to weather radars that are not dopplerized. Hence, one can only expect similar or better flood forecasting results with a technically superior radar like the WSR-88D in the US.

Such a new technology however presents certain problems or challenges, that were pointed out by the previous research works. The central question around weather radar data is its accuracy (Collier, 1996; Hudlow et al., 1991; Seo, 1998; Smith and Krajewski, 1991). Knowledge of this accuracy is of crucial importance for assessment of the effects of errors in rainfall estimates on flow forecasts. Weather radar measurements of precipitation can be improved by calibrating the precipitation computed from radar data with measured raingage data (James et al., 1993; Shedd and Fulton, 1993; Shelton and May, 1996). At the NWS, NEXRAD data is available in 3 different levels of precipitation processing, corresponding to different levels of ground-truthing with a network of raingages (see Appendix B). In addition, interference from mountains, lighting or hail can adversely affect Doppler radar performance (National Weather Service, unknown date; Reed, 1997). Some turbulence-detection systems are therefore being devised by NWS researchers.

2.6. Hydrologic modeling with NEXRAD data

As mentioned earlier, level III WSR-88D rainfall data is desirable for flood forecasting modeling and is only used by the NWS at the Oklahoma location of one of its RFCs for issuance of river forecasts. Concerning the use of NEXRAD data, the NWS hydrologic forecasting system is far from being operational at

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this point. Researchers at the Hydrologic Research Lab (HRL) of the NWS are currently studying a semi-distributed modeling approach where mean areal precipitation values are derived from NEXRAD data for each subbasin (National Weather Service Hydrologic Research Lab, 1997). The next step in their research will address the development of a gridded distributed parameter model. In the US, such modeling has just begun and, although different agencies are working on it, publications on the subject are scarce. Direct use of radar rainfall data in operational hydrologic forecasting is not yet a generalized practice. Rainfall-runoff simulation using NEXRAD rainfall data is investigated at the US Army Corps of Engineers Hydrologic Engineering Center (HEC) in California (Kull et al., 1996). An initial application of a newly devised modeling approach has been undertaken in the Illinois River watershed in Oklahoma and Arkansas using the only stage III data available to date (Peters and Easton, 1996). Since then, the Salt River basin in Missouri has been selected as a demonstration site (Hydrologic Engineering Center, 1996a). The model has also recently been developed for the Muskingum River basin in Ohio (Hydrologic Engineering Center, 1996c). Results are encouraging although it is evident that the availability of stage III data would be a marked improvement. The value of a flood-forecasting system depends on the lead time it provides for issuing warnings (Peters, 1993). A minimum lead time is needed for the system to be practical. The lead time that is potentially achievable depends on: (1) the spatial and temporal characteristics of storm rainfall and the ability to forecast these, (2) rainfall-runoff response characteristics of the watershed and the ability to simulate these and, (3) the time required to recognize and evaluate the flood threat and take appropriate action. The value of a warning depends also on its reliability. Short-term hydrologic forecasting refers to forecasts with lead times of hours to several days, and is the case of all cities located in the vicinity of a river. Hydrologic models for short-term forecasting may employ channel routing, rainfall-runoff simulation, or both (Lettenmaier and Wood, 1993). The choice of a model type depends on the required forecast lead time, the characteristics of the basin and the storm events. For example, for quick responding -flashy-watersheds, lead times are very short and a forecast of precipitation is needed because future precipitation will reach the forecasting point within the lead time. Flood forecasting models, like the one HEC is working on, could probably

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be appropriate in such cases if they can use NEXRAD data in real-time. This could possibly help ameliorate the lead time in the future.

Another important trait of the HEC model is its use of GIS. GIS are computer based systems that provide powerful data mangement capabilities for handling spatial databases. The important relationship of map information and spatial data to hydrologic analyses makes hydrology a natural field for the application of GIS (Deckers and Te Stroet, 1996; Woodbury and Jawed, 1993). In recent years, more powerful GIS tools have become available at ever decreasing costs, and data that can be used in a GIS to assist in performing detailed hydrologic analyses is rapidly becoming available through public and private efforts. In addition to these factors, the natural appeal of the graphical analysis and display of GIS data contributes to the popularity of this technology. Numerous studies describe research efforts involving hydrologic applications of GIS ranging from the synthesis and characterization of hydrologic tendences to the prediction of response to hydrologic events (DeVantier and Feldman, 1993). GIS is an emerging computer technology therefore the challenge for the hydrology discipline consists of determining how to use GIS in a useful fashion. For example, a wealth of information about the morphology of a land surface is available from DEMs. Since so much of hydrology is linked to processes on the earth's surface, the use of GIS in delineating depressions, overland flow paths and watershed boundaries is obvious (Da Ros and Borga, 1997; Jenson, 1991; O'Callaghan and Mark, 1984; Quinn et al., 1991; Tarboton et al., 1991). HEC's new approach makes use of these procedures (Hydrologic Engineering Center, 1996b). With the arrival of gridded precipitation estimates from NEXRAD, specific GIS procedures are needed for the computation of the necessary rainfall-runoff modeling parameters for streamflow forecast from the superposition of the radar grid over the watershed boundaries. In their new model, researchers at HEC have focused on writing programs to accomplish these tasks (Feldman, 1995; Hydrologic Engineering Center, 1996b). They, and the Center for Research in Water Resources in Texas, are currently pursuing their efforts in refining the set of procedures (Reed and Maidment, 1995). No other similar research work has been encountered in the published literature.

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2.7. Rainfall-runoff modeling techniques

The relationship between rainfall and runoff has been one of the central themes of hydrologic research for many years (O'Connell, 1991). With the growth of digital computing power in the 50s and 60s, an increase in hydrologic modeling activity took place and hundreds of rainfall-runoff models have been described in the literature (Loague and Freeze, 1985; Peters, 1993; Renard et al., 1982). All models seek to simplify the complexity of the real world by selectively exaggerating the fundamental aspects of a system at the expense of incidental detail (Anderson and Burt, 1985). In presenting an approximate view of reality, a model must remain simple enough to understand and use, yet complex enough to be representative of the system being studied. All hydrologic models are approximations of reality, so the output of the actual system can never be forecast with certainty. Hydrologic phenomena vary in the three space dimensions, time and randomness (Chow et al., 1988). A practical model usually considers only one or two sources of variation. Deterministic models -a given input always produces the same output- are the most commonly used in the field of hydrology. To model the complete drainage area system accurately would call for a very detailed knowledge of the basin, of the physical and biological processes governing water movement and the way that these interact. In practice, this is not feasible and simplifications have to be made (Blackie and Eeles, 1985). These can be either in the representation of the physical structure or in the representation of the processes involved. The choice of what to simplify and to what extent can be dictated by a wide range of considerations. The most common simplification made in basin modeling is lumping or spatially averaging. The implication is that the basin system, its output and response can be represented mathematically only using the dimensions of depth and time. The key factor in the successful application of lumped models is the stability of the drainage area in terms of its physical characteristics and the stability of the spatial distribution of precipitation. The aggregated empirical parameters that they contain have a complicated physical interpretation and a large range of variation (Kuchment et al., 1996). Lumped models have been the traditional approach to rainfall-runoff modeling (Beven, 1985; Chow et al., 1988; Peters and Easton, 1996). The most widely used model of this type is the HEC-1 flood model, developed by HEC. But in recent years, interest towards

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gaining a better understanding of the role of spatial variability and scale in the behavior of a hydrologic system has been growing (Beven, 1989 and 1991; Collier, 1996; Grayson et al., 1992; Klemes, 1983; Ogden and Julien, 1993; Sipavalan et al., 1987). Distributed models are also called physically based. By physically based, one means the model is firmly based on the understanding of the physics of the hydrologic processes that control the watershed response. Physically based hydrologic models are necessarily distributed because the equations on which they are defined generally involve several coordinates. The existence of numerous complex equations requiring a large amount of calculations, and thus the need for some appropriate computer systems, probably restricted the use of distributed models in the past research applications. The development of distributed modeling has been a slow and faltering process. However, a number of organizations have developed distributed models. The Hydrologic European System (SHE) model was developed in a collaboration between Denmark, France and the UK (Abbott et al., 1986a and 1986b). The Institute of Hydrology Distributed Model (IHDM) (Beven et al., 1987; Calver and Wood, 1995), TOPMODEL (Beven et al., 1979 and 1995; Coles et al., 1997; Morris, 1980) and the Japanese model mentioned earlier (Lu et al., 1996a and 1996b) are other examples.

Distributed models are believed to offer a great potential in four major areas (Beven, 1985). These are the forecasting of the effects of land-use change, movement of pollutants and sediments, effects of spatially variable inputs and outputs, and the hydrologic response of ungaged basins where no data are available for calibration of a lumped model. The spatially distributed nature of the input data of distributed models and their physically based parameter values are the major advantages. One current major concern is how to deal with the temporal and spatial rainfall variability over the basin (Coles et al., 1997; Obled, 1989; O'Connell, 1991; Ogden and Julien, 1993; Shah et al., 1996). The importance of the spatial and temporal rainfall distribution on the runoff hydrograph of a basin has been demonstrated by many studies (Foufoula-Georgiou and Georgakakos, 1991). For example, Wilson et al. (1979) concluded that the spatial distribution of rain and the accuracy of the precipitation input considerably influence the volume of storm runoff, time-to-peak, and the peak runoff of small catchments. Hamlin (1983), Nicks (1982) and Milly and Eagleson (1988) also reached similar conclusions for drainage basins of

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various sizes. NEXRAD weather radar gridded data offers a new measurement of the spatial development of precipitation, which represents a qualitative change with respect to the type of information previously available, and brings in a new perspective in modeling efforts. In that sense, distributed modeling is the only viable option to incorporate this new information into the modeling process (Cluckie et al., 1987; Garrote and Bras, 1995a; National Weather Service, 1994; Todini, 1980). Garrote and Bras (1995a and 1995b) are working on developing a distributed model for flood forecasting that can accept distributed rainfall input. The model, called Distributed Basin Simulator, is based on detailed topographic information from DEMs and the variable source area concept with a kinematic infiltration model. As mentioned earlier, Lu et al. (1996a and 1996 b) in Japan are developing a distributed hydrologic model for use with weather radar distributed rainfall data. Research work by Shah et al. (1995), with the SHE, also tries to provide ditributed rainfall inputs. The WATFLOOD model in Canada is another example of current efforts towards integrating gridded radar data in hydrologic forecasting systems (Kouwen, 1988). Documented studies on the topic are scarce in the current literature. One of the concepts behind HEC's new flood forecasting model is the use of a distributed procedure so as to accommodate the spatial nature of NEXRAD rainfall data. It seems clear that distributed models will become more used in the near future, therefore focusing on their improvement might be a wise option. One however needs to keep in mind that physically based models, even though they offer a better process representation and predictive capability of runoff modeling, are still only models (Kuchment et al., 1996). Their ability to simulate and predict the behavior of any given river basin depends on the adequacy of their representation of the basin, available data, and the computational procedures used. There is no such thing as a fully distributed model since an assumption of constant parameter values always needs to be done at some spatial level in the process.

2.8. Recap

Flood forecasting systems to reduce heavy damages caused by floodings have been in the past recent years the object of great interest among hydrologists. Although some systems are successfully operating in some basins, futher research, both theoretical and experimental, is needed. The

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traditional approach to flood forecasting has been the use of rainfall input estimated from a number of raingages driving a lumped parameter hydrologic model. An example of such application is the National Weather Service Forecasting System. The advent of new technologies such as the WSR-88D weather radar, GIS and high-speed computer workstations provides new opportunities for improved hydrologic forecasting. In particular, the high resolution gridded precipitation data from NEXRAD has prompted the need for hydrologists to reexamine existing rainfall-runoff models. The challenge appears to be the implementation of physically-based models that can take maximum advantage of the new data. Researchers at HEC in California are working on the problem. Their new semi-distributed flood forecasting model makes use of NEXRAD rainfall data and GIS. It is not completely operational yet and needs refining and improvement. This project is a collaboration work with researchers of the HEC group to develop and test the model in a flat region like Iowa. The study basin is the Squaw Creek watershed in central Iowa, which has a history of flooding. The hope is that a new technology such as NEXRAD can help guard against loss of life and property form extreme weather by increasing the lead time. This would then provide an answer to what Mark Twain used to say: “Everybody talks about the weather, but nobody does anything about it” (Reed, 1997).

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CHAPTER 3. DESCRIPTION OF THE STUDY SITE

3.1. Characteristics of the Squaw Creek basin

The Squaw Creek watershed is located in Story, Boone, and Hamilton counties in central Iowa. It is typical of watersheds that lie in recently glaciated agricultural landscapes of central Iowa and the Midwest (Prior, 1991). Because of the glaciation which occurred as recently as 14,000 ybp, the overall topography is youthful, with an incompletely developed drainage system. The watershed is part of a larger river basin, the Skunk River Basin, covering 11280 km^2, which extends southeastward to the Mississipi River (Figure 3.1) (Heinitz, 1978).

The Squaw Creek drainage area is small, covering approximately 563 km^2 (Figure 3.2). It has a narrow floodplain of about 0.5 km wide which provides little storage capacity for flood waters (Snyder & Associates Inc., 1996). The main channel is about 53 km in length. Squaw Creek is a third-order stream and drains level to gently undulating topography before emptying into the South Skunk River. Average channel slope is about 1.7 m/km (Slack et al., 1993). The creek begins in southwestern Hamilton county, flows through northeastern Boone county and northwestern Story county before reaching the South Skunk River. Main tributaries to Squaw Creek include, from north to south, Crooked Creek, Montgomery Creek, Lyndis Creek, Onion Creek, Clear Creek, College Creek, and Worrell Creek.

Most of the area was originally covered by a continuous mosaic of prairies, forests and wetlands (Thompson, 1992). These ecosystems provided a natural equilibrium for the water cycle, with extensive areas of high water infiltration resulting in low runoff (Anderson et al., 1996). After the arrival of Euro-American settlers in the mid-1800s, the original landscape was, for more than 90%, gradually converted to agricultural and urban uses. Nowadays, most of the area in the Squaw Creek watershed is in cultivation, with corn and soybeans being the major crops (Andrews and Dideriksen, 1981; DeWitt, 1984; Dideriksen, 1986). Smaller acreages of pasture, oats, hay and woodland are found as well. A small urban area is present in the basin: the city of Ames, located at the confluence of Squaw Creek with the South Skunk River, and

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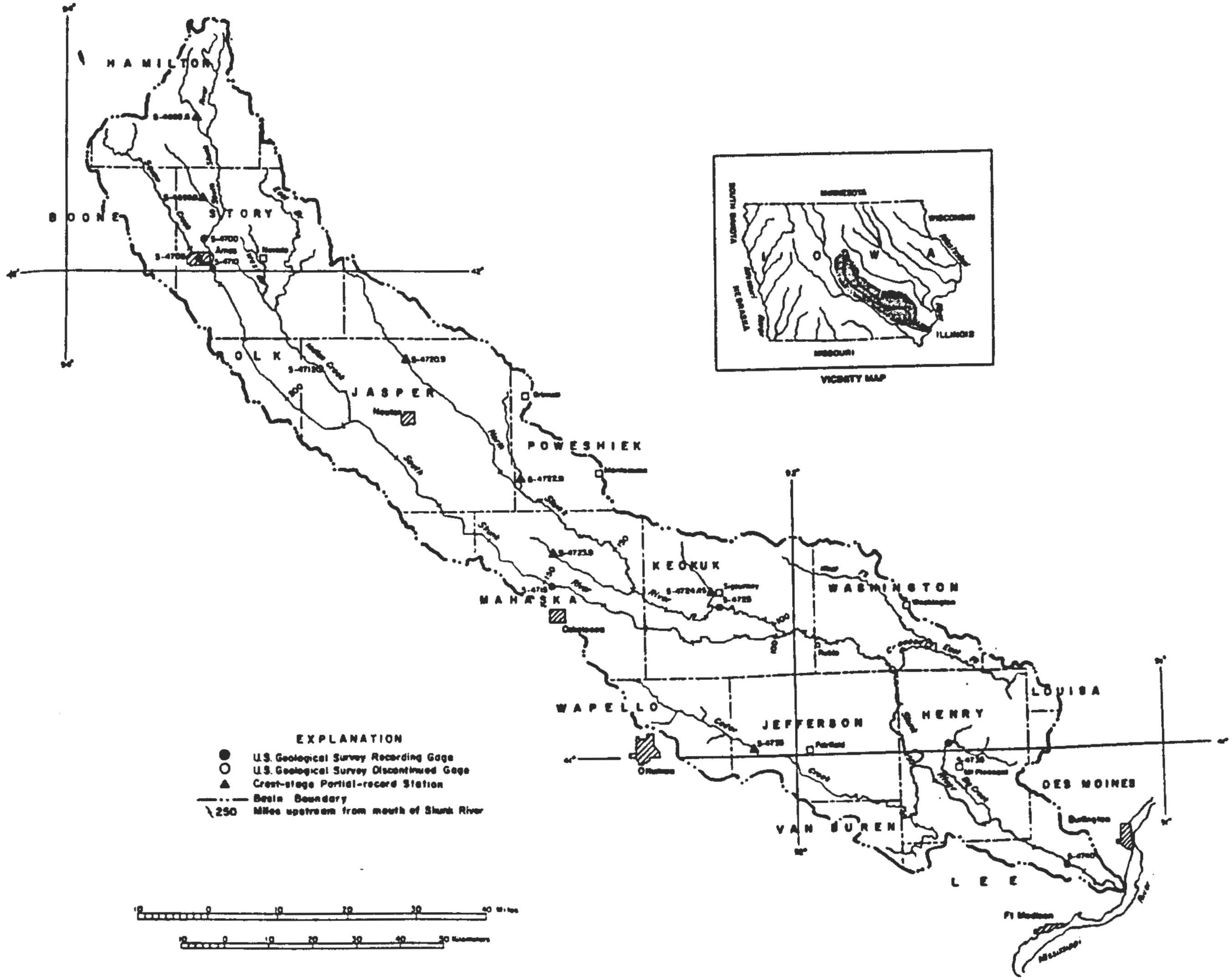

Figure 3.1. Map of the Skunk River basin

(reprinted from Heinitz, 1978, p.3)

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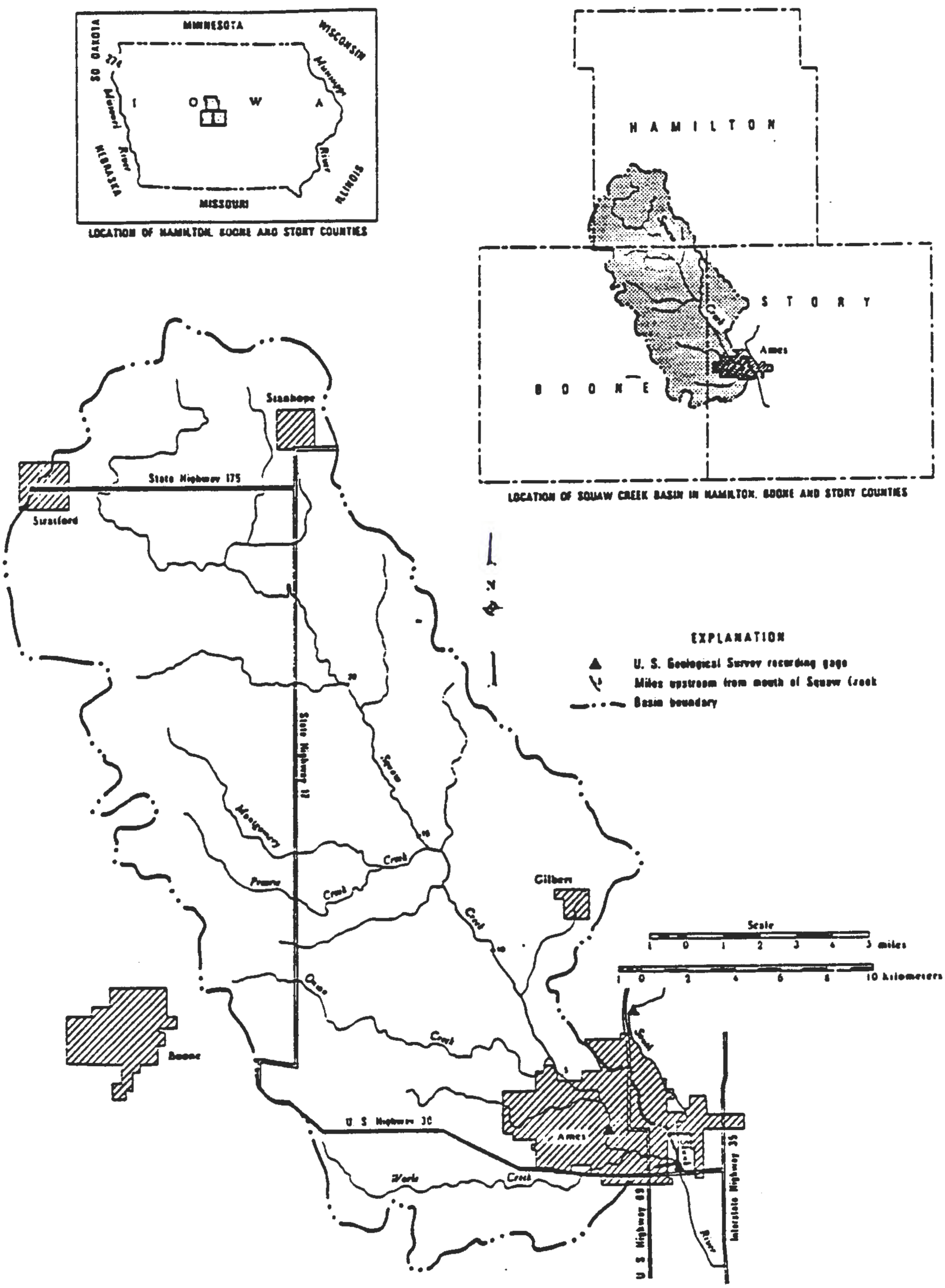

Figure 3.2. The Squaw Creek watershed
(reprinted from Lara and Heinitz, 1976, p.3)

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comprising approximately 6 km of reach on Squaw Creek (Figure 3.2). Other smaller communities are Stratford and Stanhope located in the northern portion of the basin, and Gilbert in the southeastern portion. The road network within the watershed includes US Highways 30 and 69 and State Highways 175 and 17. A major east-west railway also crosses the basin at Ames

The predominantly agricultural landscape is due to the fact that soils are particularly fertile in this part of the state. They are located within the end moraine of the last glacial advances called the Des Moines Lobe and all formed in glacial till or alluvium from till (Andrews and Dideriksen, 1981; DeWitt, 1984; Dideriksen, 1986; Thompson, 1992). The major soil associations in the Squaw Creek watershed are the Clarion-Webster-Nicolett, Clarion-Storden-Coland, Hayden-Lester-Storden, Coland-Spillville-Zook and Canisteo-Okoboji-Nicolett (Glanville, 1987). These soil groups are characterized by low elevations and a moderate relief. Because many soils are poorly drained and many wetlands represented an obstacle to cultivation and grazing, an extensive, yet unmapped, tile drainage system has been established since the beginning of this century. The presence of such subsurface tiles probably did impact the overall runoff potential within the basin and increased the flood hazard (Montgomery, 1986). In addition, channelization and excavation of drainage ditches have sped up the movement of water and created deeply incised channels with unstable banks. Modern agriculture has thus accelerated the streamflow in the landscape and made the area more prone to flooding.

Soil survey data (Andrews and Dideriksen, 1981; DeWitt, 1984; Dideriksen, 1986) and USGS water reports (Slack et al., 1993) indicate that the average annual precipitation over the basin is approximately 81 cm. Of this, about 75% usually falls between April and September. Precipitation early in Spring in conjunction with snow melt can create favorable flooding conditions.

3.2. Historical flooding in the Squaw Creek basin

Due to the drainage network, the land use, and the topography of the Squaw Creek basin, a flooding peak discharge response from peak precipitation usually occurs within only 12 hours (Snyder & Associates Inc., 1996). For nearly 100 years now, loss of life and considerable property damage has occurred because of flash flooding of the stream. Streamflow data from the US

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Geological Survey gauging station located in Ames, 3.8 km upstream from the mouth of the basin, indicate that major floods have occurred several times (Table 3.1) (David Conell, personal communication, 1997; Glanville, 1987; Tebben, 1997; US Department of the Interior Geological Survey, 1985, 1991, 1994, 1997). It is interesting to point out that these events only occur between the months of March and August, which is correlated to the precipitation pattern mentioned earlier.

Table 3.1. Notable recorded floods in the Squaw Creek Basin (greater than 115 m^3/s)

Date	Gage Height (m)[a]	Discharge (m^3/s) (Instantaneous Peak Flow)
June 4, 1918	4.42	195.39
July 17, 1922	3.26	116.95
March 1, 1965	3.26	118.93
June 27, 1975	4.27	319.98
March 19, 1979	3.60	150.08
June 13, 1984	3.95	203.31
June 17, 1984	3.89	193.12
May 19, 1990	3.23	118.93
June 17, 1990	4.87	353.96
July 9, 1993	5.64	688.10
July 13, 1993	4.24	245.22
June 17, 1996	4.66	359.62

[a] Present gage was installed in 1965. Prior to 1925, a non-recording gage was located 0.9 km upstream from the present gage at a different datum. No official gage was maintained from May 1927 to February 1965.

The first official gauge measurement of flooding in the Squaw Creek basin was recorded in June 1918. Since then, several flooding situations have been observed (Table 3.1). These situations correspond to an exceedence of the flood stage of 2.1 m (Heinitz and Wiitala, 1978). Based on the extent of the damage it caused, the flood of 1975 was described as the largest that ever occurred in the basin (Heinitz and Wiitala, 1978; Lara and Heinitz, 1976). Property damage

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was estimated to exceed $1,000,000 and one person died. The floods of the summer of 1993 however beat that record (Einhellig and Eash, 1996). The magnitude of the 1993 floods was due to a persistent wet-weather pattern throughout the Upper Midwestern US for at least 6 months preceding the event (Parrett et al., 1993; Wahl et al., 1993). Heavy rainfall events of between 5 and 13 cm fell over the basin during late June and early July 1993 (Parrett et al., 1993). Due to an average precipitation of one and one-half to two times normal -normal precipitation for 1961 to 1990- during the period from January to July 1993, the soils were completely saturated and all incoming rainfall simply became direct runoff. Ames, located near the basin outlet, experienced unprecedented flooding. Total economic loss within the community was estimated to be over $10 million (Snyder & Associates Inc., 1996). The 1996 flood was not as severe as the 1993 flood, either in terms of damage to homes and businesses, or damage to public facilities (Hoffman, 1996). Nonetheless, total damage was estimated to average $1.4 million.

3.3. Summary of past efforts in flood mitigation

Faced with the risks of flash flooding, the City of Ames has been actively involved in finding ways to mitigate flooding consequences. Federally sponsored public works were therefore considered. In the late 40s, the Rock Island District of the US Army Corps of Engineers studied potential flood control measures in the Skunk River Basin, including the Squaw Creek basin (US Army Corps of Engineers, 1971). Two reservoirs were proposed: one on Squaw Creek west of Gilbert, and the other on the Skunk River a few miles upstream from Ames. Strong opposition from local landowners forced abandonment of the projects. In the late 60s, they were re-examined and the reservoir on Squaw Creek was found to be economically justified on the basis of flood control benefits (US Army Corps of Engineers, 1971). The project was, however, again abandoned because of strong local opposition. Due to renewed public interest, the Corps undertook a reevaluation report exploring alternative measures of flood control in the Squaw Creek basin (US Army Corps of Engineers, 1987). Opposition from the Iowa Department of Natural Resources, the City of Ames, and local residents led to definitive closure of public works projects within the area.

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The City of Ames then turned to other alternatives. A study of Squaw Creek and Skunk River Basin floodplains by the US Army Corps of Engineers Rock Island District helped determine flood-prone areas (US Army Corps of Engineers, 1966). It eventually led to the development of floodplain zoning ordinances for regulation and use of the floodplain within the city limits (City of Ames, 1975). Flood damage reduction features, such as earthen berms, elevated entrances, and major under drainage systems were incorporated in the construction of the Iowa State Center Complex and Iowa State university dormitories in the 70s. Other flood control structures were put in place to try and channel flood flows away from property adjacent to Squaw Creek. Elwood Drive, a major north-south artery in Ames, was constructed at a higher elevation so as to serve as a levee during flooding events. In spite of these projects, important damages are still being sustained within these areas. During the flood of 1993 for instance, the Hilton Coliseum was heavily damaged when water filled up to the first row of parquet-level seats.

There are three areas in Ames that are primarily prone to major flooding damage. The multi-million dollar Iowa State Center Complex, with several buildings and extensive parking facilities, is one area of high risk. The commercial and business area along South Duff Avenue, southeast of the city, poses another threat. Residential areas along South Riverside, South Russell, South 4th and South 5th streets, south of the city along the creek, are also part of the flood prone zone. Development in the Squaw Creek floodplain has not been halted by the past flooding events. It actually seems to intensify with the construction of new residential apartments and business and commercial buildings. Over the past 30 years, several Iowa State University facilities have been constructed including housing (Snyder & Associates Inc., 1996). There has also been an increase in concentration of commercial development along South Duff Avenue. The potential for increased damage from a possible disastrous flooding event seems to be growing. The City Officials are thus exploring all possible flood mitigation alternatives.

In order to reduce the impacts of flooding, the City of Ames has set up a flood prediction and warning system (Grosskruger, 1993). In the event of a flooding situation, a precise plan of action takes place. Water Plant personnel monitor streamflow on Squaw Creek as well as precipitation amounts. If the water level reaches 1.8 m, is rising at 0.15m/hr, and at least 5 cm/hr of rain

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are falling, the Police notifies the City Disaster Response Team. Police are sent to the residential flood prone areas to conduct a door-to-door warning of the impending flooding. Business owners are telephoned. Sand bags and sand are provided in key areas. The Public Works Department takes care of traffic measures. Fire personnel can assist in the evacuation of local residents. Local Utilities disconnect power and gas lines if necessary. The Red Cross can also provide emergency shelter. Police also provides security for damaged properties, if any.

Despite the existence of this flood response strategy, City of Ames officials have been feeling uncomfortable with the flood prediction system. Public skepticism can arise when actual water levels are far short of predictions and result in a lack of action during an actual disaster event. Accurate and rapid forecasting within the Squaw Creek basin is needed. What was missing was a flood forecasting system. The NWSRFCs generate flow forecasts using operational hydrologic models for some specific locations along major rivers (Shedd and Fulton, 1993). Squaw Creek represents only a portion of the Skunk River Basin and is not classified as a major river. Hence, there is no flood forecasting available from the NWSRFC. Glanville (1987) developed a microcomputer based flood prediction model for the Squaw Creek basin based on the US Army Corps of Engineers Hydrologic Engineering Center's HEC-1 model. His model predicts the time of peak and peak discharge for large flood events. It was revised, verified and slightly modified in 1996-1997 (Tebben, 1997). This model, however, can only be as accurate as the data fed into it. Although the current network of telemetered rain gauges (Tebben, 1997) placed within the basin seems satisfactory under a well-distributed rainfall pattern, it may prove to be inadequate in providing representative rainfall intensities under intense convective storm events. This would, in turn, cause inaccurate predictions on peak discharge and time to peak. Radar data, on the other hand, with its spatial and temporal characteristics of rainfall measurement (Appendix B), could represent a great asset in the flooding prediction operations.

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CHAPTER 4. MODEL DEVELOPMENT APPROACH

HEC's ModClark computerized procedure is a new modeling approach that uses WSR-88D weather-radar NEXRAD data as distributed input to rainfall-runoff modeling (Hydrologic Engineering Center, 1995b). It is part of the HEC Hydrologic Modeling System (HEC-HMS), which is the Hydrologic Engineering Center's "next generation" software for precipitation-runoff modeling (Feldman, 1995 and 1996; Peters, 1995). The usual approach to rainfall-runoff simulations is the application of the unit hydrograph theory, developed in the early 30's (Clark, 1943; Bedient and Huber, 1989). Traditionally, this approach involves using spatially-averaged (lumped) values of basin rainfall and losses. Such values are commonly collected from raingage networks, which are generally sparse and provide a limited definition of the spatial variation of basin rainfall. Compared to this lumped approach, WSR-88D radars, on the other hand, have the capability to estimate the spatial distribution of rainfall, which brings in the possibility of using distributed rainfall for modeling.

One logical way to use radar data would be to subdivide the basin according to the gridded rainfall definition and run simulations at this level. However, this would result in a tremendous increase in the number of model parameters that would need to be determined. This approach would probably only be useful in some particular cases where parameters' spatial variability is large, or if a high resolution of runoff calculations is needed. Performing modeling with distributed rainfall input can be done in a simpler manner, without further subdividing existing basin representations and increasing runoff parameter information. This is what HEC has been developing. This simpler approach is an adaptation of Clark's unit hydrograph method to accommodate spatially distributed rainfall data (Hydrologic Engineering Center, 1995b).

Clark's work was centered around two main objectives (Clark, 1943). The first was to better define the inherent relationship between the unit hydrograph and flood routing. The second was to utilize this relationship to

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derive accurate unit hydrographs. His goals were that the synthesized unit hydrograph would reflect the influence of the shape of the drainage area, that the time elements of the unit hydrograph would reflect the storage capacity of the streams, and that his standardized calculation procedure would yield similar results when used by different people. Clark's technique was the first time-distributed synthetic unit hydrograph method (Kull and Feldman, 1998). It involves the application of an instantaneously applied unit of rainfall excess over a watershed. In his model, Clark employs two components: (1) a translation hydrograph to reflect the travel (lag) time required for one unit of rainfall excess (occurring instantaneously) to reach the basin outlet, and (2) a linear reservoir to represent natural storage effects (Clark, 1943). Figure 4.1 illustrates Clark's conceptual model. The translation hydrograph is used to reflect the surface runoff's time of travel. To develop it, the instantaneous unit of excess precipitation is lagged based on isochrone-delineated segments (lines of equal travel time) to the outlet, creating a time-discharge histogram. Linear reservoir routing is then used to reflect stream channel storage attenuation effects. The routing yields an instantaneous unit hydrograph (IUH). The IUH can be used to develop a unit hydrograph for any desired time step. This is done by averaging a series of IUHs lagged over the desired time interval. The two parameters of the Clark method are the time of concentration T_c (time base of the translation hydrograph) and the storage attenuation coefficient of the linear reservoir R. Both parameters have units of time. The time of concentration, T_c, is conceptualized as the time it takes for rainfall excess from the hydraulically most remote point of the basin to reach the outlet. It is a measure of lag due to time-of-travel effects without regard to storage effects. Unlike T_c, the storage attenuation coefficient R cannot be discerned simply by analyzing the physical characteristics of the watershed. R is a measure of lag due to natural storage effects.

In the HEC adaptation of the Clark conceptual model, the modified Clark method (ModClark), WSR-88D radar grid cells are superposed on the basin (Hydrologic Engineering Center, 1995b). The NEXRAD rainfall cell is considered as the basic modeling unit. Rainfall and losses are tracked uniquely for each cell. To transform rainfall excess into runoff, the modified Clark method proceeds the following way (Hydrologic engineering Center, 1995b; Kull and Feldman, 1998; Peters, 1995; Peters and Easton, 1996):

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1) Translation of rainfall excess (lag) on a cell basis by using the travel time of the cell (use of T_c);

2) Routing of the lagged rainfall excesses through a linear reservoir (use of R).

The methodology of ModClark is illustrated in Figure 4.2. Rainfall excess is computed for each cell using the rainfall and watershed data. Each cell's excess is then lagged to the basin outlet according to the cell's travel time. Next, individual lagged cell outflows are routed through a linear reservoir using R. The routing is identical to the one used in Clark's original method. The lagged and routed outflows are then summed. Baseflow is added, and the basin's outlet hydrograph is produced.

The travel time for a cell is determined using:

$$t_t = T_c \frac{t_l}{t_{l\max}} \text{, where:}$$

t_t = travel time from the cell to the basin outlet (translation lag)

T_c = basin Clark time of concentration

t_l = travel time index for the cell

t_{lmx} = maximum of the basin cells' travel time index.

The translation lag is computed using the travel time index for the cell. The definition of the travel time from a cell to the basin outlet is:

$$T = \frac{D}{V} \text{, where:}$$

T = travel time from the cell to the basin outlet

D = length of the flow path to the basin outlet

V = average flow velocity over the flow path.

The ModClark model is based upon the assumption that flow velocity is constant over a basin. Therefore, flow path length can serve as the cell travel time index. This information, for each cell, is calculated and confined in a Grid Cell Characteristics File generated by the HEC GIS GridParm procedure (Hydrologic Engineering Center, 1996b). From a DEM for the watershed and the application of an 8 direction pour algorithm, the flow path length is computed by summing the lengths of all segments along the path from the cell to the basin outlet. Cell area is also determined for DEM-based cells at a resolution of 100 m. Radar cells from the NWS HRAP grid are superposed. Their area and travel time indices are calculated by summing the areas and

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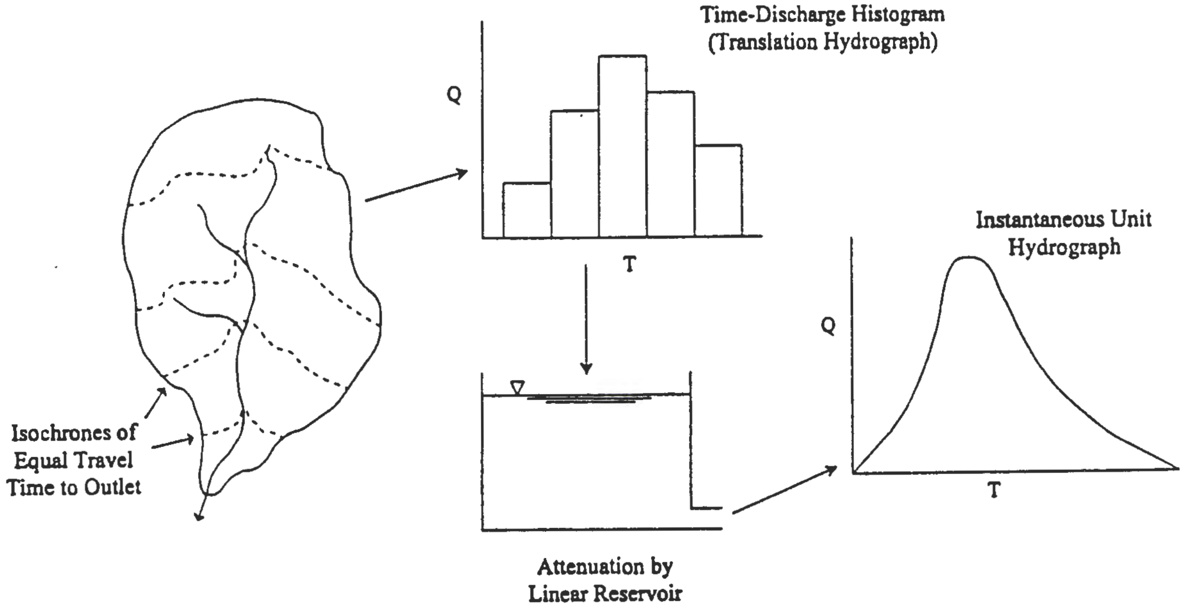

Figure 4.1. Clark's conceptual model
(reprinted from Kull and Feldman, 1998, p. 10)

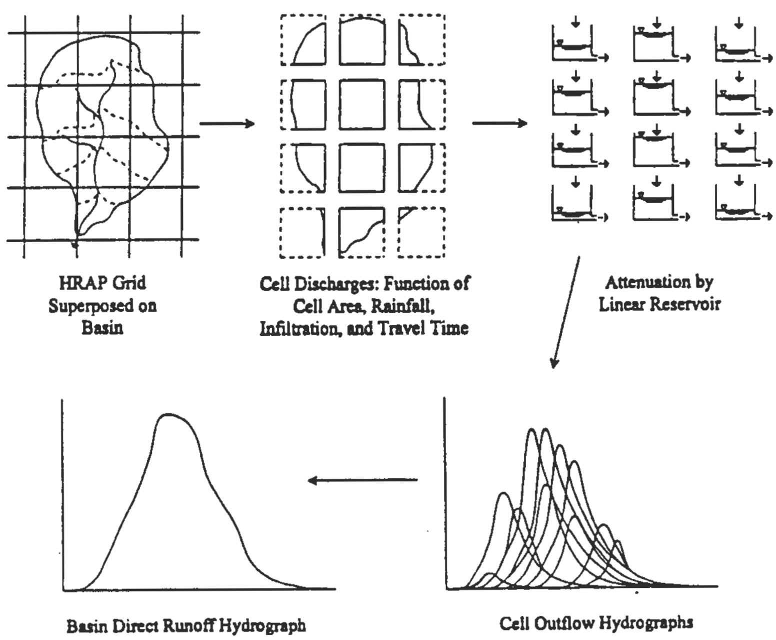

Figure 4.2. ModClark conceptual model
(reprinted from Kull and Feldman, 1998, p. 14)

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averaging the travel time indices of the encompassed DEM-based cells. These final grid cell values are considered to be constants for the basin.

Once precipitation excess for each cell is lagged to the basin outlet by the cell's travel time, it is routed through a linear reservoir according to the following equation:

$$O_i = \left[\frac{\Delta t}{R + 0.5(\Delta t)}\right] I_{av} + \left[1 - \frac{\Delta t}{R + 0.5(\Delta t)}\right] O_{i-1} \text{ , where:}$$

O_i = direct runoff at time I

R = Clark storage attenuation coefficient

I_{av} = average inflow for the time i-1 to i

Δt = time interval

O_{i-1} = direct runoff at time i-1.

These data are then utilized by ModClark. To simulate runoff from an individual basin for which grid-based hourly rainfall data is provided, infiltration and baseflow data are also necessary (Hydrologic Engineering Center, 1995b). As with the Clark parameters and the Grid-Cell characteristics file, they are entered as input within the HEC-HMS Basin Model set of data (Hydrologic Engineering Center, 1998). Subbasin loss parameters apply to all cells in the subbasin, but losses are calculated individually for each cell based on the rainfall intensity associated with that cell. The hourly NEXRAD radar rainfall data is pre-formated with the GridLoadhdp HEC program and stored in the HEC-DSS format for retrieval by ModClark within HMS.

ModClark's generated subbasin runoffs are then routed within HEC-HMS to obtain the whole basin runoff hydrograph. Necessary data for routing and combining flows are supplied to HMS with the Basin Model data set (Hydrologic Engineering Center, 1998).

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CHAPTER 5. OBJECTIVES OF THE STUDY

The goal of this particular Squaw Creek watershed study was to develop and implement HEC's new rainfall-runoff modeling procedure in the Squaw Creek watershed using NEXRAD rainfall radar data.

The specific objectives of the study were:

1- To develop the ModClark rainfall-runoff model for the Squaw Creek drainage basin;

2- To assess the accuracy of the ModClark model performance by comparison between NEXRAD radar data input, raingage data input, and actual streamflow;

3- To begin the model calibration.

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CHAPTER 6. DEVELOPMENT OF THE MODCLARK MODEL

6.1. Subbasins, flow routing and combining definition

Glanville (1987) divided the Squaw Creek basin into thirteen subbasins based on drainage data obtained from USGS topographic maps and drainage district maps. Tebben (1997) brought a small modification to the subdivisions for her study, and created two extra subbasins (Figure 6.1).

For the development of the ModClark model, the location of the two City of Ames and the USGS streamflow gages was given particular attention. Consultation with HEC (Troy Nicolini and John Peters, personal communication, 1997) led to the division of the watershed into three subbasins as most appropriate. The presence of these three gages corresponding to three subbasin outlets would enable future flood flow forecast comparisons between the ModClark model predictions and the streamflow gage data, which could be particularly useful in real-time mode. In addition, it was deemed necessary to eliminate the most southern subbasin of the original subdivision because of its location south of the USGS stream gage. Tebben's defined fifteen subbasins' division with its associated original drainage patterns was used in this study for the determination of some of the model parameters. The delineation of the watershed and subwatersheds' boundaries was further defined using GIS (see ModClark Grid-Cell Characteristics input file development) and is indicated further in Figure 6.9.

The location (latitude and longitude) of the 3 subbasins' outlets (gages) was determined from United States Geological Survey (USGS) 7.5 minute topographic quadrangle maps (Table 6.1). The flow routing schematic had to be redone, taking into account the new basin configuration. Flow routing and combining flow nodes are indicated in Figure 6.2. Subwatershed A runoff flow is routed downstream to the subwatershed B outlet and combined with the runoff flow from subwatershed B at this point. This flow is then routed to the watershed outlet and combined with the subwatershed C runoff flow.

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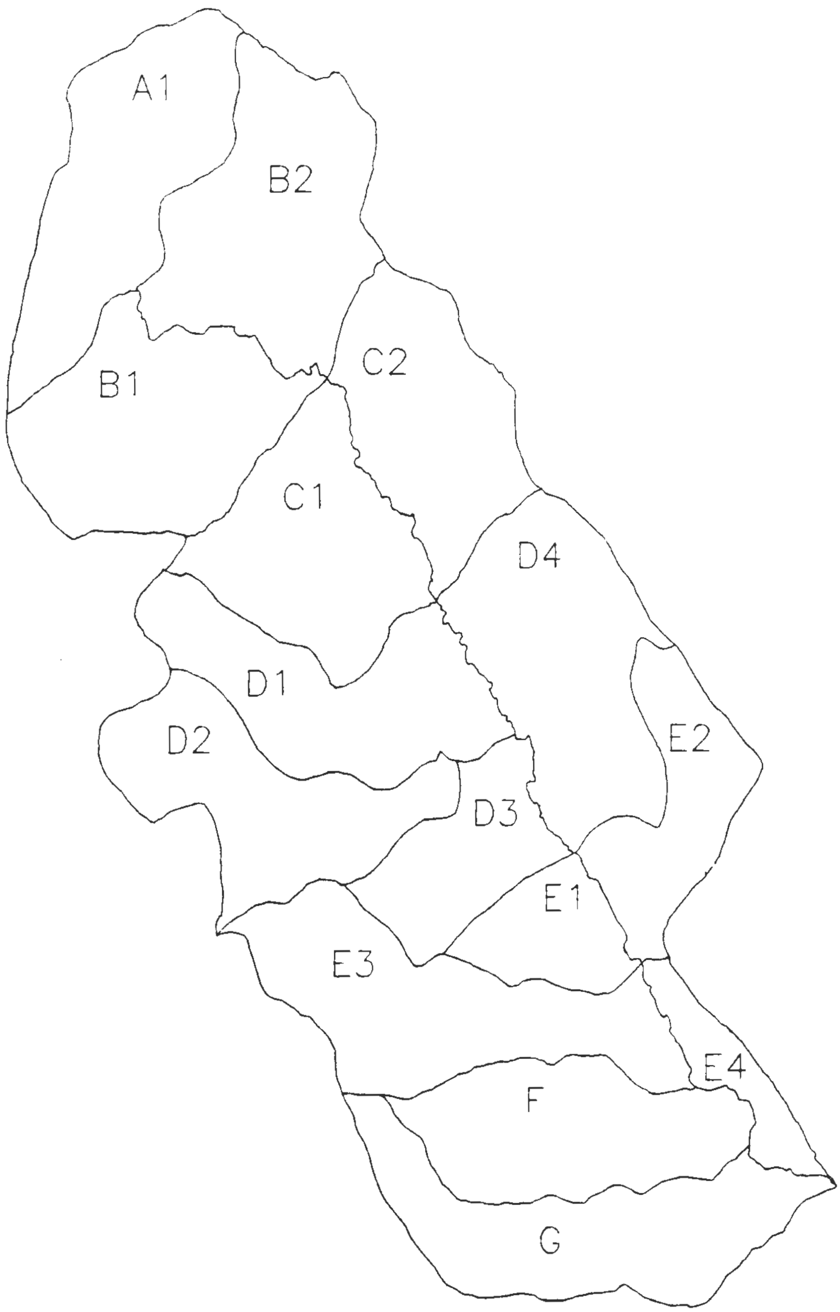

Figure 6.1. City of Ames' HEC-1 model current subbasin division for the Squaw Creek watershed (reprinted from Tebben, 1997, p.21)

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Table 6.1. Location of Squaw Creek subbasins' outlets for the ModClark model

Name	Latitude	Longitude
Subbasin A	42°09'59"	93°45'11"
Subbasin B	42°04'14"	93°40'21"
Subbasin C	42°01'14"	93°37'49"

6.2. Estimation of Clark parameters

Several methods were attempted for the determination of the Clark time of concentration, T_c, in each of the three subbasins of the Squaw Creek watershed. A regional regression analysis was performed in the first place. This technique involves correlating T_c with physical basin characteristics such as drainage area, watercourse slope, or watercourse length. Based upon the original subdivision into fifteen subbasins in the City of Ames' HEC-1 model (Tebben, 1997), average channel slope was determined in each subcatchment. Measurements were made on USGS topographic maps at 1:24,000 covering the study area using the following formula (Lara, 1973):

$$S_{av} = \frac{E_{85} - E_{10}}{0.75L} \text{ , where:}$$

S_{av} = basin average slope, in ft/mi

E_{85} = elevation of channel at a location 0.85L upstream from the basin mouth, in ft

E_{10} = elevation of channel at a location 0.10L upstream from the basin mouth, in ft

L = length of channel, in mi

Drainage area and SCS dimensionless unit hydrograph lag time for each of the fifteen subbasins were obtained from existing data (Tebben, 1997). Programs were written in SAS (Statistical Analysis Package) language and run in the SAS system to perform a regional regression analysis using values of drainage area, channel length, average channel slope and time lag. The goal was to develop an equation allowing the computation of the time lag. Several different combinations of the independent variables were tried, including a regression analysis of only drainage area and channel slope (Holder, 1985; Kleinbaum and Kupper, 1978). None of the equations obtained through either linear regression or multiple linear regression was judged

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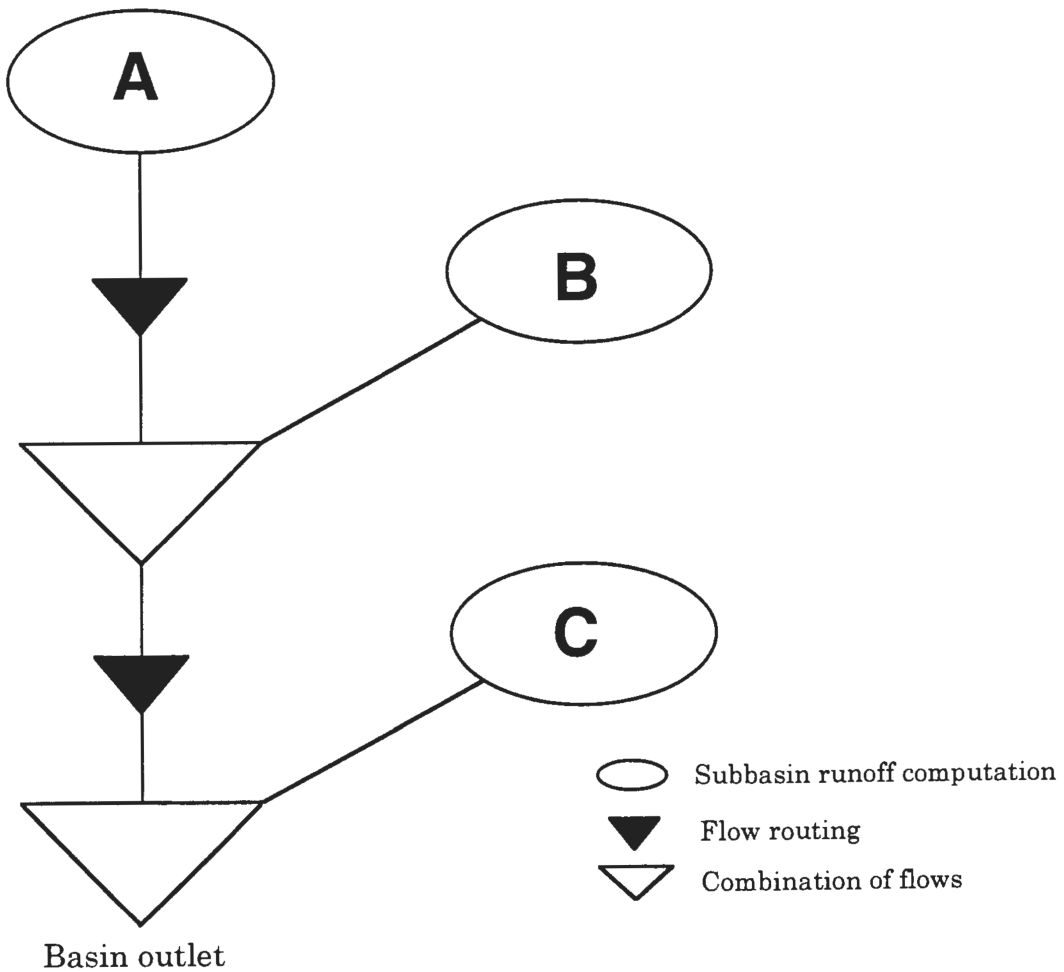

Figure 6.2. Flow routing and combining schematic for the Squaw Creek watershed ModClark model

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satisfactory in terms of the time lag value given for each of the three subbasins of the ModClark model. Subsequent use of HEC-1 (Hydrologic Engineering Center, 1990) in optimization mode for transforming the lag values into Clark times of concentration was out of the question.

A second method was thus used to determine the Clark parameter T_c. It is well known in the hydrology field as the TR-55 method and uses an analysis of physical basin characteristics (Soil Conservation Service, 1986). This procedure allows to calculate the time of concentration and travel time in a drainage area. It is based upon the assumption that water moves through a watershed as sheet flow, shallow concentrated flow and open channel flow. Travel time is the time it takes water to travel from one location to another in a watershed. It is a component of the time of concentration, which corresponds to the time for runoff to travel from the hydraulically most remote point of the watershed to a point of interest within the watershed. The TR-55 method is based on the fact that the time of concentration is computed by summing all the travel times for consecutive components of the drainage conveyance system. Sheet flow in the headwater of streams is flow over about 300 ft plane surfaces in the proximity of the basin divide. Manning's kinematic solution is used to compute the travel time:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(s)^{0.4}}$$, where:

T_t = travel time, in hr

n = Manning's roughness coefficient

L = flow length, in ft

P_2 = 2-year, 24-hour rainfall, in in

s = slope of hydraulic grade line, in ft/ft

Manning's roughness coefficient was obtained from a TR-55 table. P_2 was estimated from the Weather Bureau Technical Paper 40 (Hershfield, 1961). Values of L and s came from measurements on USGS topographic quadrangle maps at 1:24,000.

After a maximum of 300 ft, sheet flow becomes shallow concentrated flow (Soil Conservation Service, 1986). The average velocity for this flow is a function of watercourse slope and type of channel. A TR-55 figure is designed

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to determine such a value. For slopes less than 0.005 ft/ft, the following equation for an unpaved surface is:

$$V = 16.1345(s)^{0.5}$$, where:

V = average flow velocity, in ft/s

s = watercourse slope, in ft/ft

Travel time for the shallow concentrated segment is then computed from:

$$T_t = \frac{L}{3600V}$$, where;

T_t = travel time, in hr

L = flow length, in ft

Flow length and watercourse slope were measured on USGS topographic quadrangle maps.

Open channels are supposed to begin where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on USGS quadrangle sheets. Manning's equation is used to estimate the average channel flow velocity (for full bank elevation):

$$V = \frac{1.49(r)^{2/3}(s)^{1/2}}{n}$$, where:

V = average flow velocity, in ft/s

r = hydraulic radius (equal to a/P_w), in ft

a = cross sectional flow area, in ft^2

P_w = wetted perimeter, in ft

s = channel slope, in ft/ft

n = Manning's roughness coefficient for open channel flow

Manning's n value was obtained from a standard hydrology textbook (Chow et al., 1988). Estimates of channel geometry in the Squaw Creek watershed were based on previous data (Glanville, 1987). Channel slope and flow length were measured on USGS topographic quadrangle maps. Using average flow velocity, travel time was then computed with the same equation used with the shallow concentrated flow segment.

The TR-55 method was applied individually to each of the three subbasins of the Modclark model. The most remote tributary was used for the computation of the time of concentration, considering each subbasin as a whole basin in

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itself. Most tributaries in the Squaw Creek watershed show some intermittent portions. During rain events leading to floods, these intermittent parts become full. Therefore, the total length of the creek water system used in the channel flow computations included both intermittent and permanent portions. Intermittent creek portions were also assumed to have a smaller cross sectional area than permanent ones. Both were treated separately for the channel flow computations. Stream flow length used for these calculations ended at the boundary of the original subbasin (City of Ames configuration) containing the most remote tributary in question. Starting from that point, the travel time of the flow, down to the subbasin mouth, was represented by the K value (in hr) of the Muskingum routing previously developed by Tebben, 1997. In the Muskingum method, a commonly used hydrologic river routing, the K coefficient, is the time of travel of the flood wave through the channel reach (Chow et al., 1988). For subbasins B and C, this value was reduced to the corresponding length of the main channel after the tributary joining point. The overall estimate of the time of concentration for each subbasin was then obtained by adding values of travel time for each of the different components of flow. Results are indicated in Table 6.2.

Table 6.2. Clark time of concentration for the Squaw Creek subbasins

Subbasin	Time of concentration (hrs)
A	13.1
B	12.8
C	10.5

To develop estimates of the Clark storage coefficient, R, existing data from a study done by the US Army Corps of Engineers Rock Island District in the Squaw Creek watershed was used (US Army Corps of Engineers, 1987). This report was a general reevaluation study in which alternatives to the withdrawn Ames Lake project were being explored starting in 1984. Smaller reservoirs, levees, non-structural methods, soil conservation practices and channel modifications were considered. In particular, interior drainage within the Squaw Creek basin was studied. Unit hydrographs for several interior basin areas were computed with the HEC-1 flood package (Hydrologic Engineering Center, 1990) using the Clark unit hydrograph technique. Clark

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parameters were thus developed for these areas. Based upon the fact that the ratio of $R/(T_c+R)$ is relatively constant within a basin, the following value was adopted (Hydrologic Engineering Center, 1990; Thomas and Benson, 1970; Troy Nicolini, personal communication, 1997):

$$\frac{R}{T_c + R} = 0.43$$

For each subbasin of the ModClark model, T_c was replaced by its value found earlier and the equation solved for R. Reasonable estimates of R were thus obtained (Table 6.3).

6.3. Determination of other basin rainfall-runoff parameters

6.3.1. Basin and subbasin areas

Squaw Creek basin and subbasin areas were obtained in two different fashions: from the work done on the existing City of Ames' HEC-1 model, and from the GIS basin and subbasin delineation procedure (see ModClark Grid-Cell Characteristics input file development). The areas obtained with both procedures are indicated in Table 6.4.

Table 6.3. Clark storage attenuation coefficients for the Squaw Creek subbasins

Subbasin	Storage attenuation constant (hrs)
A	9.9
B	9.7
C	7.9

Considering the published drainage area for the USGS Ames streamflow gage for Squaw Creek of 528.36 km^2 (Slack et al., 1993), the decision was made to use the non-GIS areas. The flatness of the terrain makes it difficult to evaluate such drainage areas with a lot of accuracy. The difference between the numbers is therefore not great. HEC-HMS's technical features are also designed to deal with such differences during the simulations using a GIS-derived Grid-Cell Characteristics file.

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6.3.2. Loss rate parameters

Abstractions, or losses, correspond to the difference between an observed rainfall hyetograph and its associated excess rainfall hyetograph. Losses are primarily water absorbed by infiltration into the soil, with some allowance for interception of precipitation on vegetation above the ground, and depression storage on the ground surface as water accumulates in hollows over the surface. Interception and depression are assumed to be negligible in a large

Table 6.4. Comparison of areas obtained from the HEC-1 model and from the GIS basin analysis (in km^2)

Subbasin	HEC-1 model	GIS analysis
A	223.90	229.99
B	194.80	208.28
C	101.40	102.93
Total	520.10	541.20

storm (Chow et al., 1988). The Soil Conservation Service (currently the Natural Resources Conservation Service) has developed an empirical method for computing abstractions from storm rainfall. The agency has been able to relate drainage characteristics of soil groups to a curve number, CN. The CN is a function of the ability of soils to infiltrate water, land use, and the soil water conditions at the start of a rainfall event (Antecedent Moisture Conditions, AMC). Because estimates of CN are easily obtained from SCS published tables using soil type and land use data that are readily available for most watersheds, the SCS method has always been used in the Squaw Creek basin modeling work. CN values have been established on the basis of most soils comprised in the B hydrologic soil group with row crops as the major land use pattern, which is typical of soils in midcentral Iowa (Glanville, 1987; Tebben, 1997). The SCS loss rate method was thus used here as well.

For modeling purposes, the precipitation loss was considered to be a subbasin average value, assumed to be uniformly distributed over the entire subbasin. Values from the lumped HEC-1 model developed for the City of Ames could then be used. Research work by Tebben (1997) has indicated that, most of the time, a CN associated with an AMC between AMC II (normal conditions) and AMC III (wet conditions) more closely matches the HEC-1

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predicted flows with the observed ones. It is possible to use a CN corresponding to AMC II.5 because AMC and CN represent the physical reality of soil moisture. True soil moisture content is not easily represented by discrete values. At any given time, actual moisture conditions could fall somewhere between the values given for any discrete designation. Midcentral Iowa, where the Squaw Creek watershed is located, is characterized by low average precipitation from November to March because of a predominance of cold dry winds from continental Canada (National Oceanic and Atmospheric Administration, 1998). The period of April to August sees most of the annual precipitation. Evapo-transpiration is low in winter and highest in July and August (National Climatic Data Center, 1995). Soils that are soaked with moisture during winter are thus prone to a lot of surface runoff when the raining season starts in April. The majority of surface runoff occurs during the period of May to July. Using SCS CN values for modeling that correspond to AMC between II and III is therefore adequate. They were used both in the lumped model runs using raingage data and in the distributed ModClark model runs. For the latter runs, loss parameters apply to all cells in the subbasin, but losses are calculated individually for each cell based on the rainfall intensity associated with that cell. Values for the three ModClark model subbasins were obtained from an averaging of values of the original Squaw Creek basin subdivision (Troy Nicolini, personal communication, 1997). They are indicated in Table 6.5 for different AMCs.

Table 6.5. Squaw Creek ModClark model subbasin SCS curve numbers

Subbasin	AMC I	AMC I.5	AMC II	AMC II.5	AMC III
A	63	72	80	86	91.5
B	62	71	79	85	91
C	60.5	69.5	78	84.5	90.5

6.3.3. Runoff transformation parameters

Runoff transformation parameters allow the transformation of precipitation excess to direct runoff using a unit hydrograph. For the development of the model for Squaw Creek, the unit hydrograph was specified in terms of parameters defined by Clark, T_c and R, for the simulations using

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raingage data as input. Estimated values of these parameters are indicated under "Estimation of Clark parameters". These values were also used for the treatment of basin runoff as quasi-distributed when using NEXRAD radar data in the simulations. For such gridded precipitation input, the ModClark procedure also requires the specification of a cell parameter file for the development of the basin direct-runoff hydrograph (see ModClark Grid-Cell Characteristics input file development).

6.3.4. Baseflow parameters

Total streamflow is generally divided into two parts: direct runoff, and baseflow. A stream carries baseflow during most of the year and it comes from the groundwater. This subsurface flow comes from rainfall infiltrated into the basin that reaches the stream. Subsurface flow from groundwater usually accounts for flow in a channel during periods of little or no rainfall. Looking at low flow seven day data for a recurrence interval of ten years for the period of June to July for Squaw Creek indicated that values range between 0.07 and 0.002 m^3/sec (Lara, 1979), which represents a very small flow. Groundwater accretion from any storm is released over an extended period of time. Therefore, a particular storm contributing to direct runoff is not directly concerned with the baseflow. Baseflow contributions can be considered minimal for areas where there is low infiltration (Bedient and Huber, 1989). The Squaw Creek watershed's dominant land use is row crops, over which the infiltration rate is not the highest. Ames' urbanized area can also be classified as mainly impervious. For large storm events, direct runoff will completely dominate the peak of the hydrograph and baseflow contribution becomes unimportant (Bedient and Huber, 1989). The baseflow value for all the simulations for Squaw Creek was thus set to be zero since it is negligible.

6.3.5. Routing parameters

Hydrologic routing deals with the movement of a flow wave down a channel and the associated change in timing or attenuation of the wave. As the wave passes through a river reach, the peak of the outflow hydrograph is usually attenuated and delayed due to channel resistance and storage capacity (Bedient and Huber, 1989; Chow et al., 1988). The Muskingum method is a commonly used hydrologic routing method for handling a variable discharge-

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storage relationship. This method models the storage volume of flooding in a channel by a combination of wedge and prism storages. During the advance of a wave, inflow exceeds outflow, resulting in a wedge of storage. During the recession, outflow exceeds inflow, resulting in a negative wedge. There is also a constant prism of storage. The volume of prism storage is equal to KQ, where K is a proportionality coefficient, and the volume of wedge storage is equal to KX(I-Q), where X is a weighting factor having the range $0 \le X \le 0.5$. Total storage is the sum of both components:

$S = [XI + (I - X)Q]$, where:

S = channel storage

I = inflow rate

Q = outflow rate

K = Muskingum storage time constant
(time of travel of flow wave through channel reach)

X = Muskingum weighting factor

Glanville (1987) developed values for the Muskingum parameters K and X for the Squaw Creek basin. They were also used by Tebben (1997) in her modeling work. It was thus decided to use the Muskingum routing method for the modeling of Squaw Creek with the ModClark procedure. Values of the parameters for the three subbasins were derived from the existing data and are indicated in Table 6.6. For routing purposes, HMS requires the specification, like in HEC-1, of a number of subreaches (Hydrologic Engineering Center, 1990; Hydrologic Engineering Center, 1997). The number of subreaches corresponds to the travel time divided by the time interval of the input rainfall data.

Table 6.6. Muskingum routing parameters for the Squaw Creek ModClark model

Reach	From	To	Subreaches	Routing parameters	
				K	X
1	Subbasin A	Subbasin B outlet	4	4.4	0.2
2	Subbasin B	Basin outlet	4	4.3	0.2

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6.4. Development of the HMS schematic for the basin

HMS's modeling concept revolves around a specific basin schematic for the watershed in question (Hydrologic Engineering Center, 1998). A basin model consists of hydrologic elements connected with each other following defined rules. Seven types of hydrologic elements are available: subbasin, river reach, junction, reservoir, diversion, source, and sink. For the development of the HMS schematic for the Squaw Creek basin, only the subbasin, river reach, and junction elements were needed. In HMS's modeling approach, a subbasin is an element that produces a discharge hydrograph at its outlet. A river reach is conceptually a linear element for which there is a known discharge hydrograph at its upstream end, and which produces a discharge hydrograph at its downstream end. A junction is a location where two or more inflow hydrographs are added together to produce an outflow hydrograph. The HMS basin schematic for the Squaw Creek watershed is presented in Figure 6.3.

The basin schematic can include a map background showing basin and subbasin boundaries, and the stream system. HEC-HMS has the ability to display this background map on the basin schematic screen. The data necessary to draw the map is stored in a text file and needs to be specified in the basin model before hydrologic elements are placed and connected to form the basin schematic. HEC has developed an automated method to aid in the creation of the background map. The method utilizes GIS techniques and requires Arc/Info, a specific macro, and the Unix system utility awk. To create the background map for the Squaw Creek HMS basin schematic, data sources included the GridParm output (see ModClark Grid-Cell Characteristics input file development) to form the subbasin boundary map, and the stream USGS joined DLGs output (see ModClark Grid-Cell Characteristics input file development) to form the stream map. The map background can be seen on Figure 6.3.

6.5. Raingage weights and temporal distribution determination

Four NWS raingages are located in or within kilometers of the Squaw Creek basin: Ames 8WSW (in the basin), Ogden, Story City, and Webster City. Hourly precipitation data is available for these gages from the National Climatic Data Center monthly publications. Such data have previously been used by Glanville (1987) and Tebben (1997) in their modeling work. To obtain

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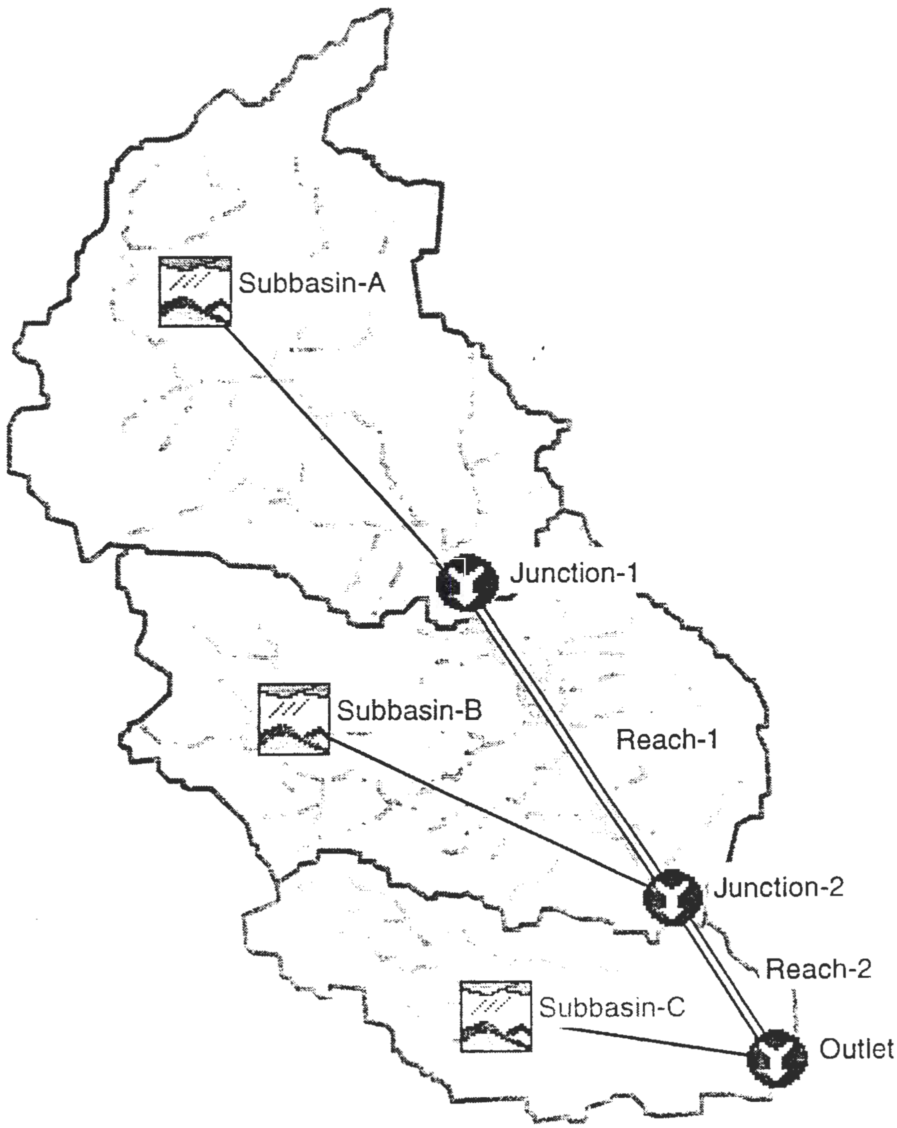

Figure 6.3. Squaw Creek basin HMS schematic and background map

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an average storm precipitation for each basin, they both calculated an areally weighted average of measurements from these raingage stations. Areal weighting factors for each subbasin were determined from drawing a Thiessen net and calculating the area of influence for each gaging station in every subbasin. The Thiessen method that they used assumes that at any point in the basin, the rainfall is the same as that at the nearest gage so the depth recorded at a given gage is applied out to a distance halfway to the next station in any direction. The relative areal weights for each gage are determined from the corresponding areas of application in a Thiessen polygon network and the boundaries of the polygons being formed by the perpendicular bisectors of the lines joining adjacent gage stations. These values represent, for each subbasin, the polygon areas under the influence of each of the four gages. A new basin and subbasin definition having been adopted for the ModClark modeling purposes, the Thiessen raingage weighting factors had to be recalculated. The Thiessen net is shown in Figure 6.4, and the calculated raingage weights used for the simulations are indicated in Table 6.7.

Besides the gage weighting, HEC-HMS's rainfall-runoff modeling approach requires the specification of the temporal distribution of the rainstorm data. Rainfall temporal distribution usually follows the one of the recording gage. In the case of the Squaw Creek basin, the four raingages are recording. Under such circumstances, the choice of the raingage "best" at reporting the temporal pattern of distribution of a rainstorm had to be made from a judgement based on the location of the gage and the general temporal distribution of this raingage for all storms (Troy Nicolini, personal communication, 1998). The Story City station was judged "best" and it was decided to distribute each storm rainfall in time using the temporal pattern of incremental rainfall data from that gage. A temporal distribution gage weight of 1 was thus used for the Story City gage, and 0 for the other gages for entry in the HMS basin model.

6.6. ModClark Grid-Cell Characteristics input file development

To develop the grid-cell parameter file for input to the ModClark model, HEC's GridParm procedure was used. This software is a set of procedures using Arc Macro Language (AML) and FORTRAN programs for evaluating ModClark runoff parameters for NEXRAD radar grid cells from USGS DEMs

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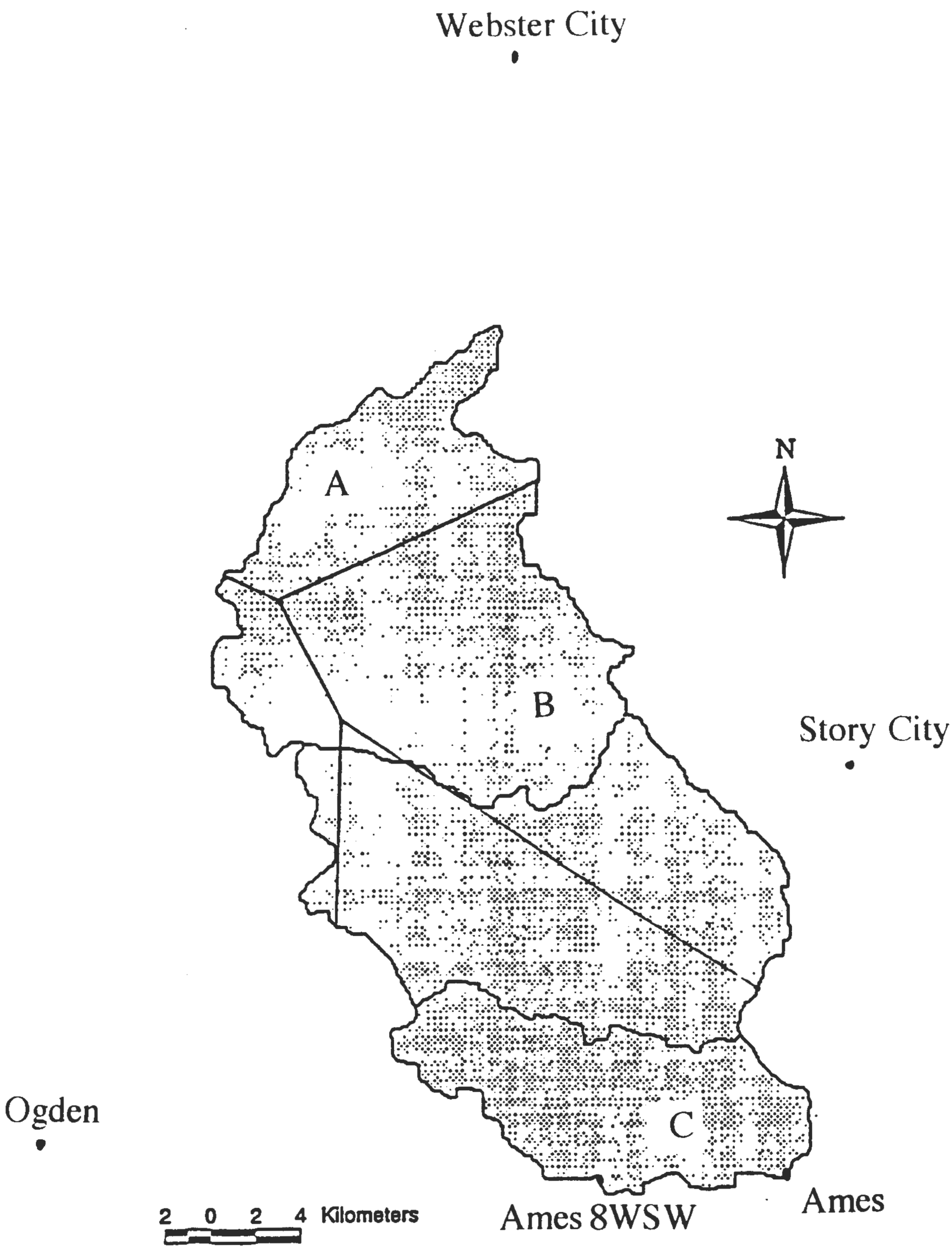

Figure 6.4. Thiessen raingage weights for the Squaw Creek basin

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Table 6.7. Storm Thiessen raingage weights for the ModClark model

Subbasin	Webster City gage	Ogden gage	Ames 8WSW gage	Story City gage
A	0.303	0.1	0.006	0.591
B	0	0.04	0.6	0.36
C	0	0	1	0

(Hydrologic Engineering Center, 1996b). It requires Arc/Info GIS Version 7.0 or higher with the Grid module running on a Unix System, the Unix utility gunzip, and Internet access. Having a FORTRAN 77 compiler can also be useful when certain FORTRAN programs need to be corrected and recompiled to fit a different Unix platform. The ModClark parameters computed by GridParm include, for the whole basin, cell identification from the HRAP coordinates, cell area from the intersection of the NEXRAD grid with the watershed boundaries, and average flow travel length from cell to subbasin outlet. The major steps of the GridParm procedure are summarized in Figure 6.5. DEM data is first processed so as to delineate the stream network. Watershed and subwatersheds are subsequently located. Values of hydrologic parameters are then calculated from the intersection of the set of watershed boundaries with the cells of the NWS precipitation-reporting grid.

Prior to using GridParm, a number of system setups, including setting default paths and variable values, had to be performed in Unix for the sake of organization and workability, due to the large amount of data being generated during the procedure.

HEC provided Arc/Info coverages of the hydrologic unit maps covering the Iowa region at a scale of 1:250,000 (Figure 6.6). These data provide an excellent reference frame to approximate the spatial extent of data needed in a hydrologic study. The only method that GridParm provides for defining the limits of a study area is the selection of a hydrologic catalog unit. Hydrologic catalog units come from the USGS series of hydrologic unit maps, which present the boundaries, numerical codes, and names of river basins in the US (Seaber et al., 1987). Hydrologic units are identified by a unique code consisting of two to eight digits based on the four levels of classification in the Hydrologic

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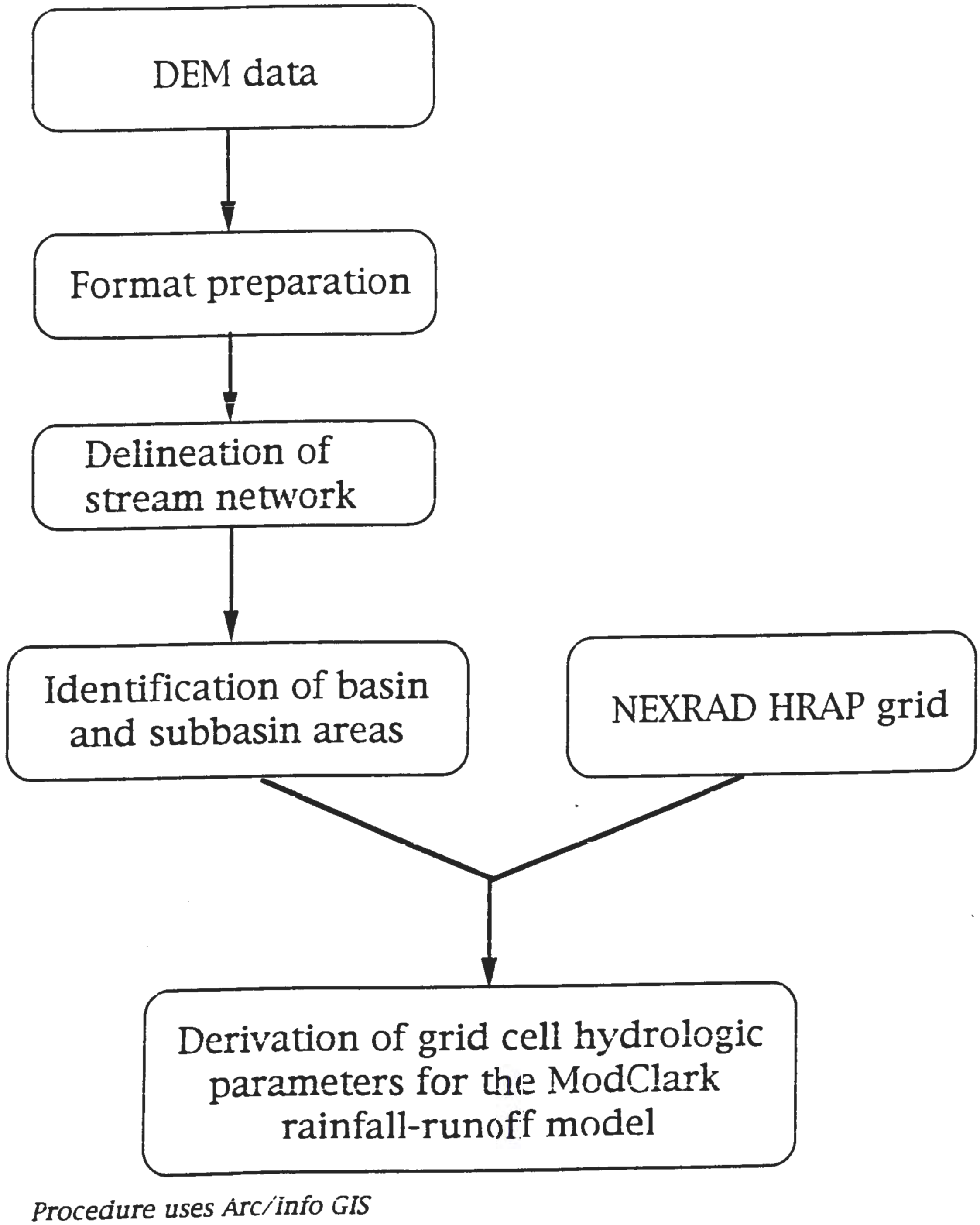

Figure 6.5. HEC GridParm procedure

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Figure 6.6. Hydrologic Unit map coverage of the study area in Iowa

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Unit Code system (HUC): region (two digit), subregion (four digit), accounting unit (six digit), and catalog unit (eight digit). Catalog units represent the finest scale resolution of this HUC and corresponds to the smallest drainage areas. From an identification of the catalog unit containing the study area, the DEM quadrangles needed for the analysis were determined. The Squaw Creek basin is located within the catalog unit 07080105 (South Skunk) (Figure 6.7). Due to the small size of the watershed, it was determined that only one USGS DEM quadrangle was needed to cover the entire basin: Waterloo-West. It was obtained by file protocol transfer (ftp) over the Internet from the USGS EROS Data Center (EDC). A 1-Degree DEM, or 3 by 3 arc-second data spacing, provides elevation data coverage in a 1 by 1 degree block and has a grid resolution of approximately 90 m^2 (US Geological Survey, 1990). Two 1-Degree quadrangles provide the same coverage as a standard USGS 1x2 degree map sheet at 1:250,000. Three arc-second DEMs are created by the Defense Mapping Agency and distributed free of charge by the USGS over the Internet.

Extracting data from the DEM file, after decompression using the Unix gunzip utility, was the next step. In order to process digital elevation data for a hydrologic study, it must first be projected into a flat map coordinate system so that the coordinates are measured in units of distance rather than degrees. This is necessary because the GIS functionality for computing area, distance and slope depends upon the data being in a cartesian coordinate system. An equal-area projection is appropriate for hydrologic modeling because drainage area on the globe is preserved in the projected space. Precipitation depth-volume relationships are then preserved (Reed and Maidment, 1995). An Arc/Info macro called "demload" was run to convert the data to a usable Arc/Info Grid format (Hydrologic Engineering Center, 1996b). After projection to an equal-area coordinate system (Albers equal-area conic projection), a new grid was created by the resampling of the elevation data from the original grid at 100 m spacing, which is common practice for data derived from 3 arc-second DEM. The 100 m cell grid was subsequently processed for removal of sinks which are cells having an elevation lower than surrounding cells. Sinks are errors in data due to the resolution of the data or the rounding of elevation numbers to the nearest integer value. Data resampling during the projection also creates artificial sinks. Sinks should be filled to ensure proper delineation of basins and streams. If they are not filled, a derived drainage network may

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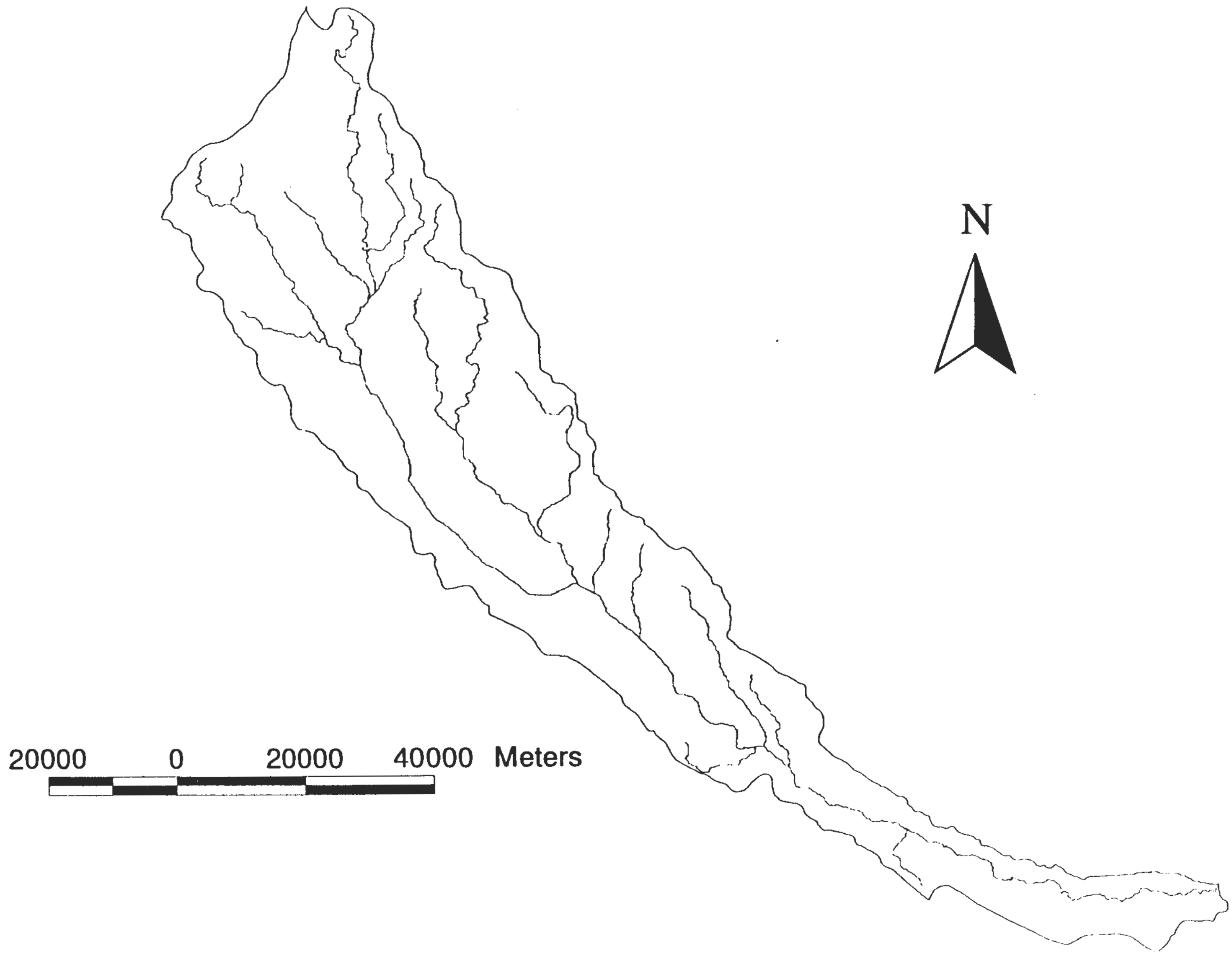

Figure 6.7. Location of the Squaw Creek basin within the catalog unit

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be discontinuous. The removal of sinks creates what is known as a "hydrologic" DEM. Arc/Info Grid functions were then used by the program to delineate the stream network. A flow direction and a flow accumulation need to be assigned to each grid cell. The Flowdirection function (8-direction algorithm) assigns a unique value to a cell indicating to which of its 8 neighboring cells it will flow -assuming water can only flow in 1 of these 8 possible directions (Figure 6.8). In the output grid, numbers are assigned according to the following convention: E=1, SE=2, S=4, SW=8, W=16, NW=32, N=64 and NE=128. An output grid of flow directions, or flow direction grid, was then obtained. A drainage network was formed from this flow direction data, using the number of cells upstream of any given cell as the flow accumulation value. The Flowaccumulation function takes the flow direction grid obtained as input and computes for each output cell the cumulative total number of cells draining into that cell. A flow accumulation grid was then obtained. Cells with no flow accumulation are on the watershed boundary, whereas those with high flow accumulation value are considered to be stream cells. The stream network derived differs depending on the threshold value of flow accumulation used to identify streams, with a smaller threshold yielding a denser stream network. Here, cells collecting flow from 100 km2 or more were identified as stream cells to obtain the stream grid.

From the information on the latitude and longitude of each of the three gages in the basin (Table 6.1), a data file containing these locations in decimal degrees was created. It was used by the Arc/Info macro called "demwsh" in the process of delineating the watershed subbasins. A point coverage of these outlet locations was generated and projected from geographic coordinates into the same Albers projection coordinate system as the DEM. To delineate basins, the outlet points needed to be grid cells that lie on the grid stream network. DEM-derived stream location and gage location did not correspond, primarily because they come from different data sources. To delineate watersheds correctly, the gage points must fall precisely on the gridded streams delineated from the DEM. Using the gage-location display of the program allowed for each subbasin outlet point location to be manually adjusted to the cell in the DEM-derived stream network closest to the gage location. A grid of the adjusted outlet locations was then created. To delineate a watershed for each stream reach, the cells at the downstream end of each

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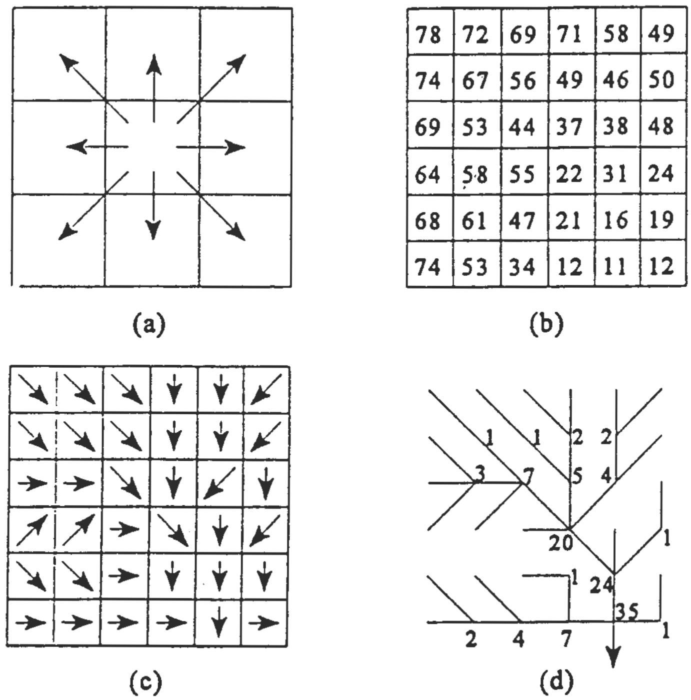

Figure 6.8. The 8-Point Pour Down Method: (a) Eight possible flow directions; (b) A grid of elevations; (c) Corresponding grid flow accumulation; (d) Resultant flow network with cell flow accumulations. (reprinted from ESRI, 1994)

reach needed to be identified as outlets. Each of these outlet cells needed to have a unique value. Ultimately, the unique value for a given outlet would be assigned to all the grid cells in the watershed delineated from that outlet. Given the gridded stream network and the flow direction grid, the Grid function Streamlink produced an output grid of the stream network so that each cell in a given stream reach contained a unique value. From this grid and the grid of outlet locations, a new grid containing only the outlet points, each with its unique location attribute inherited from the gage outlet file, was generated by the program using the Grid Selectpoint function. Next, with the Grid Watershed function, the program identified areas that drain to the specified outlet points (the gages) by using the flow direction grid and the new gage location grid. Subwatersheds are defined by the program as cells that drain to the adjusted gage locations cells. A polygon coverage of the delineated watersheds was then generated from the grid of delineated watersheds for

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subsequent use with the radar grid coverage. In Arc/Info, the development of the drainage network allows the possibility of measure of the flow distance from any cell in a subwatershed to the outlet of the subwatershed. The Grid function Flowlength was then used to compute, based on the flow direction grid and for each DEM cell in each subwatershed, the length along the drainage network to the subwatershed outlet.

Finally, the "parmhrap" Arc/Info macro was run to create the ModClark parameter file. A rectangular grid of HRAP cells covering the study area was first created. The geographic coordinates for the corners of the cells were determined using a grid definition subprogram. This HRAP cell grid was then transformed into a polygon coverage in the Albers projection and overlaid on the delineated watershed and subwatershed boundaries polygon coverage. The resulting polygon coverage was then converted to a grid format for calculation of hydrologic parameters. These were calculated for each newly defined cell resulting from the overlay. The NEXRAD rainfall cell being taken as the hydrologic response unit, its properties were estimated by averaging the corresponding properties of the approximately 1600 DEM cells present within the rainfall cell. Using the Grid function Flowlength, the average travel flow distance from each cell center to the subbasin outlet was computed by the program by averaging the flow lengths of all DEM cells within the NEXRAD cell boundaries. Areas of the intersected polygons were obtained from the resulting polygon attribute table. The Grid-Cell Characteristics file containing the values of these parameters was then generated.

A comparison of the stream delineation, obtained from the DEM with GridParm, with the RF1 stream representation in the GIS data provided by HEC (Environmental Protection Agency (EPA) 1:500,000 Digital Line Graph (DLG) representation of stream systems in the US called River Reach Files (RF1)) showed some discrepancies. The errors were due to the DEM representation of a very flat topography like the one in central Iowa. In such a flat terrain, even the slightest elevation error can lead to a considerable change in the determination of flow directions by the program. Therefore, it was decided to manipulate the DEM before re-running the GridParm procedure. The purpose was to ensure proper stream locations so that the flow directions would be more accurate, hence resulting in a more accurate watershed delineation.

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A procedure called a stream burn-in was therefore attempted. It used 1:100,000 Digital Line Graphs (DLGs) stream network Arc/Info coverages from the Iowa Department of Natural Resources obtained by ftp over the Internet. These coverages represent a dense network of streams. The procedure consisted of overlaying the stream data onto the DEM. Any DEM grid cell which had a stream arc passing through it was then lowered by a certain elevation. From HEC's previous burn-in experience with the Muskingum River basin (Hydrologic Engineering Center, 1996c), it was known that a deep burn-in results in better stream location. The burn-in was thus attempted at a depth of 100 m to ensure that the channel system would be clearly defined. The manipulation was done in Grid in Arc/Info using the merged stream DLGs of Story, Boone and Hamilton counties. The combined DLG line coverage was then projected in Arc from its UTM coordinate system into the same coordinate system as the DEM: Albers. This ensured that both data sets could coincide for Arc/Info to align them correctly. The obtained burnt-in DEM coverage, with its sinks filled, was then used to re-run the GridParm procedure so as to obtain a new grid cell characteristics file. The programs were launched at the point of use of this data in the whole process. Because the dense stream network represented by the DLGs was able to "force" the previously errant contributing areas to flow in the proper direction, a satisfactory watershed delineation was obtained this time (Figure 6.9). The coverage resulting from the intersection of the HRAP cell grid with the watershed boundaries is indicated in Figure 6.10. This second version of the gridcell parameter file was retained for use in the ModClark model (see Appendix A). The output ASCII file lists, for each cell, the HRAP cell x and y coordinates (south west corner), the average travel distance from the cell center to the outlet, and the grid cell area, all grouped by subbasin.

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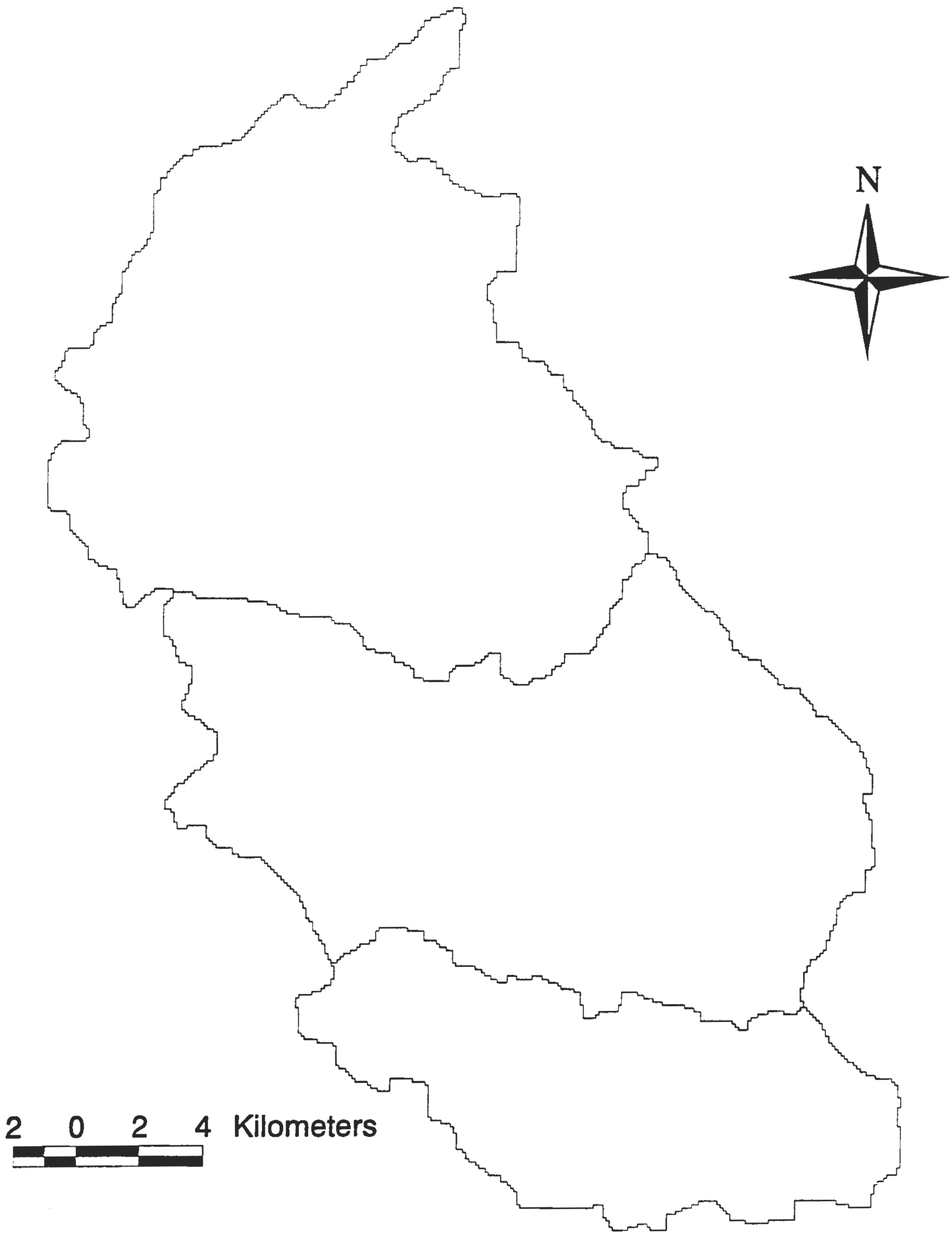

Figure 6.9. Squaw Creek watershed and subwatershed delineation from DEM and DLGs using HEC's GridParm procedure

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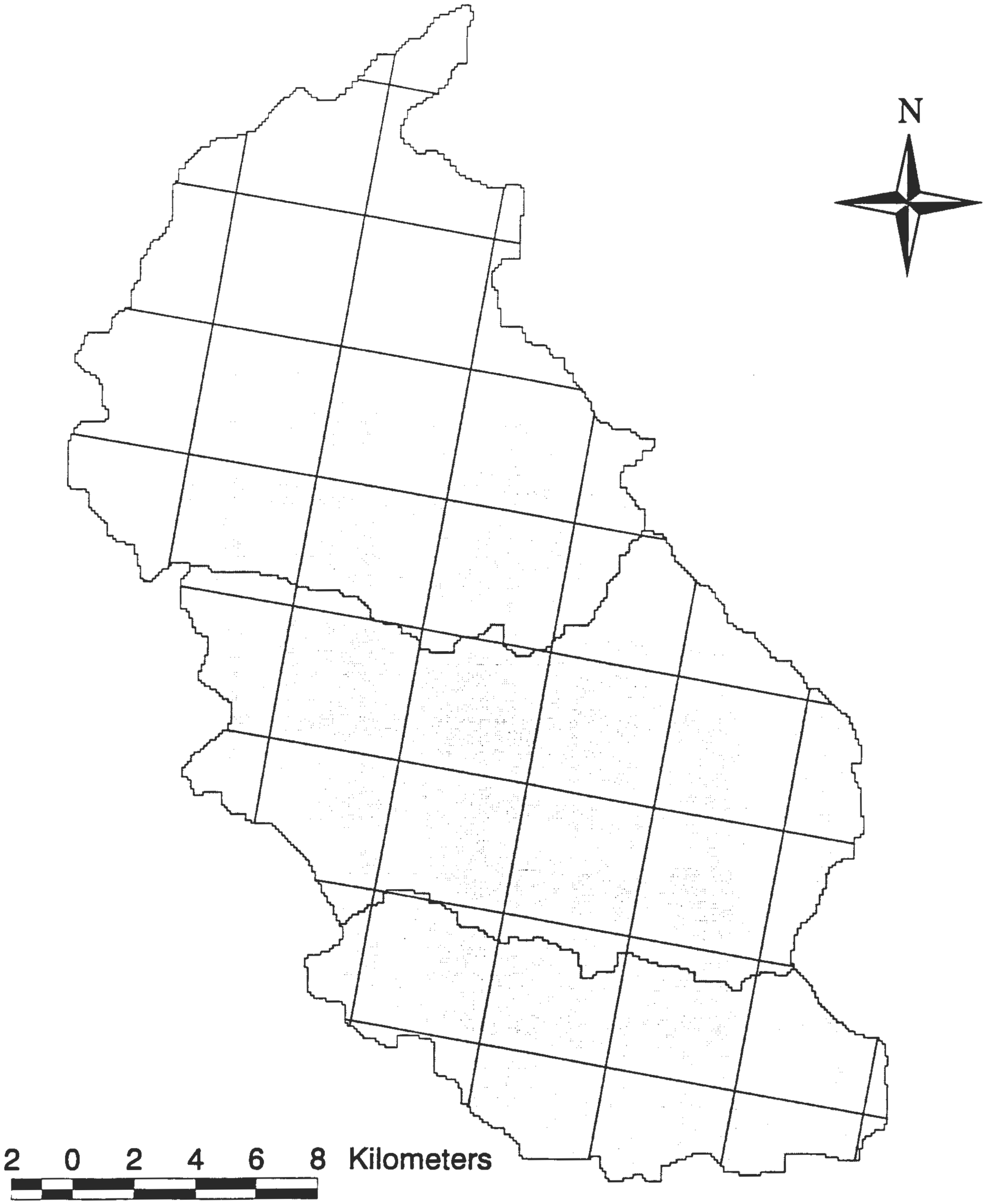

Figure 6.10. Squaw Creek watershed boundaries intersected with the NWS HRAP grid using HEC's GridParm procedure

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CHAPTER 7. VERIFICATION AND ADJUSTMENT OF THE MODCLARK MODEL

7.1. Approach

To verify and test the ModClark model, historical precipitation data was used. Three rainfall events were chosen. For the Squaw Creek basin, the closest NEXRAD radar site is the one located in Johnston, IA, near Des Moines. The radar having been installed in 1995 (Karl Jungbluth, personal communication, 1997), the choice of historical events was restricted to after this date. For rainfall-runoff modeling with HEC-HMS, the intent was to choose rainfall events that are spatially varied (different raingages reporting different rainfall amounts), yet temporally centered (one major peak of rainfall) (Troy Nicolini, personal communication, 1997).

The selection of events took these considerations into account. It began by a preliminary analysis of the flow data obtained from the USGS for the Water Year 1996 (see Streamflow data) and was followed by a study of the corresponding raingage data (see Raingage data). Consultation with the contact person at the NWS office in Johnston brought more information on the possible selected dates (Karl Jungbluth, personal communication, 1997). On June 16-17, 1996, flooding occurred in the Squaw Creek basin in Ames, following a heavy rainstorm. This event was therefore of interest. On July 16-17, 1996, 51 to 76 mm of rain fell on the basin, with heavy rains in the southwest part of the drainage area. This event was also deemed interesting to use for testing the model. Following about 125 mm of rain over the basin, the NWS issued a flood warning on July 24, 1997, which was later found to be the result of an overestimation. This rainstorm was thus also selected.

For each storm, the objective was to compare HEC-HMS simulation results obtained with gridded precipitation data input on one hand, with simulation results using raingage precipitation data input on the other hand. HMS runs with gridded rainfall data (quasi-distributed modeling) used ModClark as the runoff transformation method. Runs with raingage data (lumped modeling) utilized the Clark unit hydrograph method so as to provide a similar basis for

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comparison. Results were also compared to corresponding observed streamflow data.

An additional case event, the Flood of 1993, was chosen for study. The NEXRAD system was not in place at the time, so there was no possibility of testing the ModClark model using this historic flood. However, it appeared interesting to be able to verify the Clark lumped model for this event using raingage data. One underlying reason was the notoriously severe aspect of the Flood of 1993 (National Weather Service, 1994). Another reason was to test the ability of HEC-HMS to respond to storms that are temporally close to one another. The first half of July 1993 was studied because it is when flooding records were broken, causing extensive and unprecedented damage in Ames. Three consecutive rainfall events were recorded between July 8 and July 13 and were modeled for flood prediction.

7.2. Data acquisition and management

7.2.1. Raingage data

Early 1997, the City of Ames installed five raingages in the Squaw Creek basin, in an effort to improve its flood prediction system. Data from these gages was not available for the 1996 events. Data for the July 1997 event were retrieved using the archival system but presented many missing blocks of data, which prevented any reliable use for input to the Clark model here. All cases thus used NWS raingage data as input. Hourly raingage precipitation data for the four NWS raingages of interest for the Squaw Creek basin were obtained from monthly publications by the National Climatic Data Center (NCDC) (National Data Climatic Center, 1993, 1996b, 1996c, and 1997a). With the Utility Program DSSTS in HEC-DSS (Hydrologic Engineering Center, 1995a), the raingage data corresponding to each storm was then used to create DSS data files for input to HEC-HMS.

7.2.2. Streamflow data

Hourly streamflow data for the USGS stream gage No. 05470500 for Squaw Creek at Ames was obtained by ftp from the USGS Hydrologic Office in charge of the gage maintenance in Fort Dodge, IA, after consultation with the USGS headquarters in Iowa City, IA. The data was obtained in SHEF format (Standard Hydrologic Exchange Format) for the Water Year 1996, for July

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1997, and for July 1993. Using a complex utility program, it was subsequently converted to DSS format by HEC for use with HMS.

7.2.3. Radar data

The acquisition of NEXRAD radar data was not as easy as the previous data, and turned out to be a quite problematic task. The NWS office in Johnston did not have NEXRAD data in digital format and could not procure it from the RFC in Minnesota, whose responsibility includes the Squaw Creek basin area. Contact with a weather products vendor company in Colorado indicated the possibility of purchasing a CD ROM containing hourly precipitation data. But this data originated from raingage readings and not from NEXRAD sites. An investigation of possible Internet sources of data led to the finding of archived level II NEXRAD data distributed by the NCDC for all existing NEXRAD sites in the US (National Climatic Data Center, 1997c). This hourly precipitation data is provided on 8 mm tapes. Through collaboration with the NWS office in Johnston, the level II data tapes corresponding to the chosen rainstorm dates were obtained directly from the NCDC. The Johnston team routinely uses a software package called WATADS to display Archive II data (Karl Jungbluth, personal communication, 1998). It however has no means of retrieving the data from tapes in digital format, which is what was needed for input to the model. Since HEC was better equipped in terms of software systems, it was then decided to ship the tapes from Iowa to California where HEC would examine how the data could be extracted. Investigation by a HEC specialist led to the conclusion that this archived II data could not be used. The format of the data was uncommon, and HEC had no software that could read the information on the tapes. Contacts between HEC and the NCDC in North Carolina were not helpful either, for NCDC does not have an operating software capable of converting tape II data into digital format. NEXRAD radar data is still a new type of data, and technology and uniformity of data have not yet caught up with it, especially from a modeling standpoint. Modeling with NEXRAD data is very uncommon in the US (see Chapter 2), which accounts for the great difficulty in locating and and obtaining such data in the digital format necessary for input to the model.

An alternative solution was thus needed to get data. Based on the fact that hourly radar images can be printed with WATADS, it would be feasible to

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produce such print outs for each hour of each selected rain event for the Squaw Creek drainage area. The NWS contact person in Johnston agreed to help with the procedure. Using a transparency of of the HRAP grid output over the basin obtained from the GIS application of GridParm, it appeared possible to superimpose it on each print out to make manual readings of precipitation for each cell. An input file of hourly radar precipitation data could then be created using the coordinates of each cell as identified in the grid-cell parameter file obtained with GridParm, and converted to a DSS format with the help of HEC. In the meantime, HEC succesfully contacted the RFC in Minneapolis, and found out it was possible to obtain Stage I NEXRAD radar data in digital format. This type of data was fine since HEC has devised a software called GridLoadhdp which is capable of reading level I and converting it into a DSS format. This was definitely a breakthrough in the process of obtaining radar data since the "manual" solution would have been long and tedious, though feasible.

7.3. Event simulations and results

The Modified Clark Runoff Simulation Program (ModClark) was originally developed as an individual program to simulate runoff from individual basins for which grid-based hourly rainfall data is provided (Hydrologic Engineering Center, 1995b). A complete basin-wide ModClark model required the additional use of HEC-DSS for NEXRAD data retrieval and storage of subbasin discharge hydrographs (Hydrologic Engineering Center, 1995a), and of HEC-1 for routing and combining the flows (Hydrologic Engineering Center, 1990). The intention was to use ModClark in this context for the Squaw Creek basin model. During the research work, HEC's Research Division, which is working on the HEC NexGen project (see Chapter 4), came up with the first PC Beta version of HEC-HMS. HEC-HMS has the capability to utilize gridded rainfall via the integration of ModClark in its technical features. HEC's Research Division then decided the Squaw Creek project would be a Beta tester for HEC-HMS, before the official release of the software. Different updates of the Beta version, followed by the official version 1.0 released in April 1998, were thus used for the simulations to test the developed Squaw Creek ModClark model. During the research work, different software malfunctions were

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detected and reported to HEC. Using HEC-HMS for the Squaw Creek basin thus contributed to the improvement of this program.

7.3.1. Model input and event simulation

HEC-HMS provides a variety of options for simulating rainfall-runoff processes (Hydrologic Engineering Center, 1997). Unit hydrograph and hydrologic routing options are similar to those in HEC-1. Among the new capabilities is the addition of ModClark. HMS is comprised of a Graphical User Interface (GUI), integrated hydrologic analysis components, data storage and management capabilities, and graphic and reporting facilities. The GUI is used for the specification of basin components, input of data for the components, and viewing of results. HEC-DSS serves for the entry, storage, and retrieval of time series and gridded data.

The basic framework for runoff simulation is similar to that in HEC-1, with computations performed in an upstream-to-downstream sequence. The execution of a simulation requires the specification of three sets of data. The first, labeled Basin Model, contains parameters and connectivity data for the hydrologic elements. The second set, labeled Precipitation Model, consists of meteorological data and information required to process it. The third set, labeled Control Specifications, specifies time-related information for a simulation. A run is configured with one Basin Model, one Precipitation Model, and one Control Specifications.

Specific input files originally developed for use with the overall ModClark river basin analysis methodology were discarded because they could not be utilized in the HMS context, which is totally different. The values of the different parameters estimated for these input files were however retained. HEC-HMS rainfall-runoff simulations with raingage data input were done in a lumped mode. The methods used for loss rates calculation, precipitation transformation, and flow routing were the SCS CN, the Clark Unit Hydrograph, and the Muskingum routing, respectively. DSS data files of raingage precipitation and observed flow were also utilized. Simulations with Stage I radar precipitation data input were performed in a distributed mode. NEXRAD data being in Universal Coordinated Time (UTC), a time shift of minus 5 hours was applied so computations could be performed on a local time scale. SCS CN, ModClark, and Muskingum routing were the methods used

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for loss rates, rainfall transformation, and flow routing, respectively. DSS data files containing gridded data and observed flow data were needed. The distributed mode also required the specification of a grid-cell parameter file developed using the GIS GridParm procedure.

7.3.2. Results

<u>7.3.2.1. Lumped Clark model</u>

<u>*7.3.2.1.1. June 1996 flood event*</u>*:*

For the June 16-17 1996 event, the AMC parameters to use in the model were set to AMC III, the wet soil condition. This was chosen after a review of climatological conditions in central Iowa since the beginning of the 1996 year. An unusual amount of snowfall in January resulted in an extensive snowmelt early spring (National Climatic Data Center, 1996d). In addition, the month of May was extremely and unusually wet (National Climatic Data Center, 1996a and d). In view of this data, AMC III conditions seemed most appropriate.

Results from simulations with HMS for the June 1996 event predicted a peak flow of 317.14 cms on June 17th at 09:00 AM (Figure 7.1). According to the USGS rating curve for the streamflow gage at Squaw Creek on Lincoln Way in Ames (Appendix C), this discharge would result in a river stage of about 4.6 m, which corresponds to flooding -- the flood stage being of 2.1 m (Appendix C).

The USGS streamflow gage failed during the event. The flow data obtained for these dates could however be used because they had been corrected with observer readings (Alvin R. Conkling Jr., personal communication, 1998). Such data indicated an observed peak of 359.62 cms at 10:00 AM on June 17th, corresponding to a river stage of 4.75 m above the flood stage. The HMS prediction was thus close to reality.

The City of Ames used their HEC-1 model for that particular event and obtained a fairly good prediction (Tebben et al., 1997). Import of their HEC-1 input file into HEC-HMS showed a computed flow peak of 278.09 cms at 10:30 AM on June 17th, which corresponds to a river stage of 4.42 m, above the flood stage (Appendix C). This HEC-1 simulation used an AMC of I.5. The HMS flow computation, though using a different AMC, was within the same range as the City of Ames' prediction.

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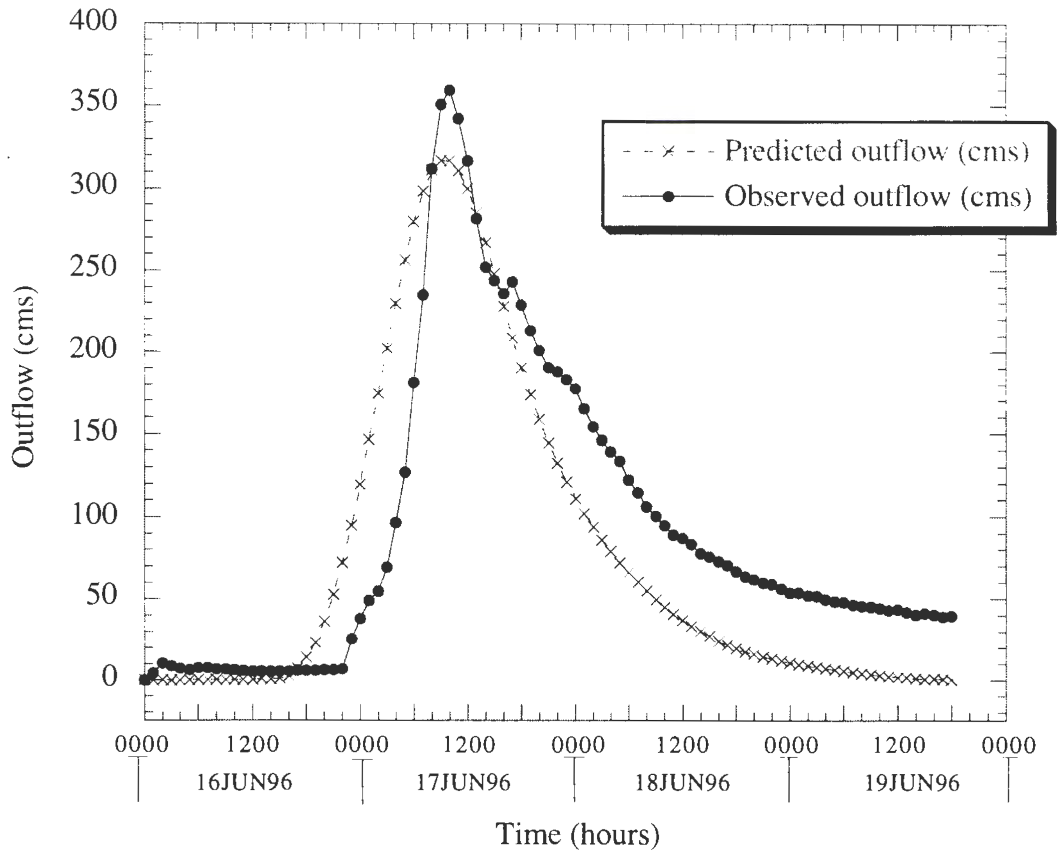

Figure 7.1. Simulation for the June 1996 flood using raingage data as input

7.3.2.1.2. July 1996 high water event:

The choice of soil moisture related parameters was not easy for the 16-17 July 1996 event. This event occurred exactly one month after the flood of June 96, which could translate into wet soil conditions. There was basically no rain between these two events (National Climatic Data Center, 1996b and c), and the conditions for water evaporation were very favorable (National Climatic Data Center, 1996d). So it was deemed possible to use an AMC of II or even I.

Results from both these simulations are shown in Figures 7.2 and 7.3 and indicated an overprediction. It was obvious that there was a large discrepancy between the observed and the computed river flows: 28.32 cms for the actual peak versus 240.69 cms and 144.61 cms for the computed under AMC II and I conditions respectively.

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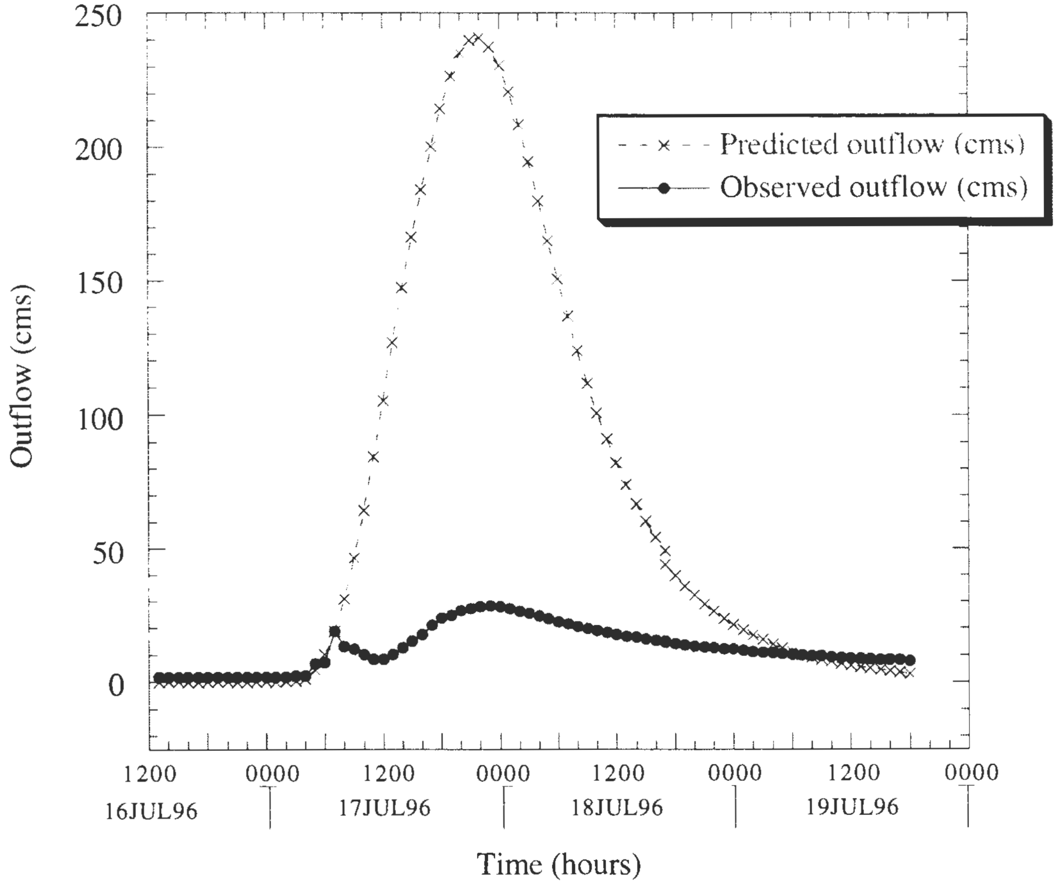

Figure 7.2. Simulation for the July 1996 event using raingage data as input (AMC II)

It was interesting to note that the timing of the discharge peak of the computed hydrograph was very close to the peak time on the observed hydrograph. The prediction was at 09:00 PM on July 17 for the computed hydrograph, and at 10:00 PM on the same date for the observed.

7.3.2.1.3. July 1997 high water event:

The year of 1997 was a dry year in central Iowa (National Climatic Data Center, 1997b). Overall, less than average precipitation fell. Modeling with loss rate parameters corresponding to AMC I thus seemed adequate.

HMS simulation results showed a large discrepancy between the observed flow and the computed flow, with an obvious overprediction (Figure 7.4). The

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observed peak discharge was 37.01 cms, and the predicted peak flow 127.38 cms.

Although slightly smaller, the difference in flow magnitude for this case was similar to the one of the July 1996 event. As with this event, one could observe that the timing of the discharge peal of the computed hydrograph was very close to the peak time of the observed hydrograph. The observed peak flow was at midnight on July 24. The computed peak flow was to occur at 11:00 PM on July 24th.

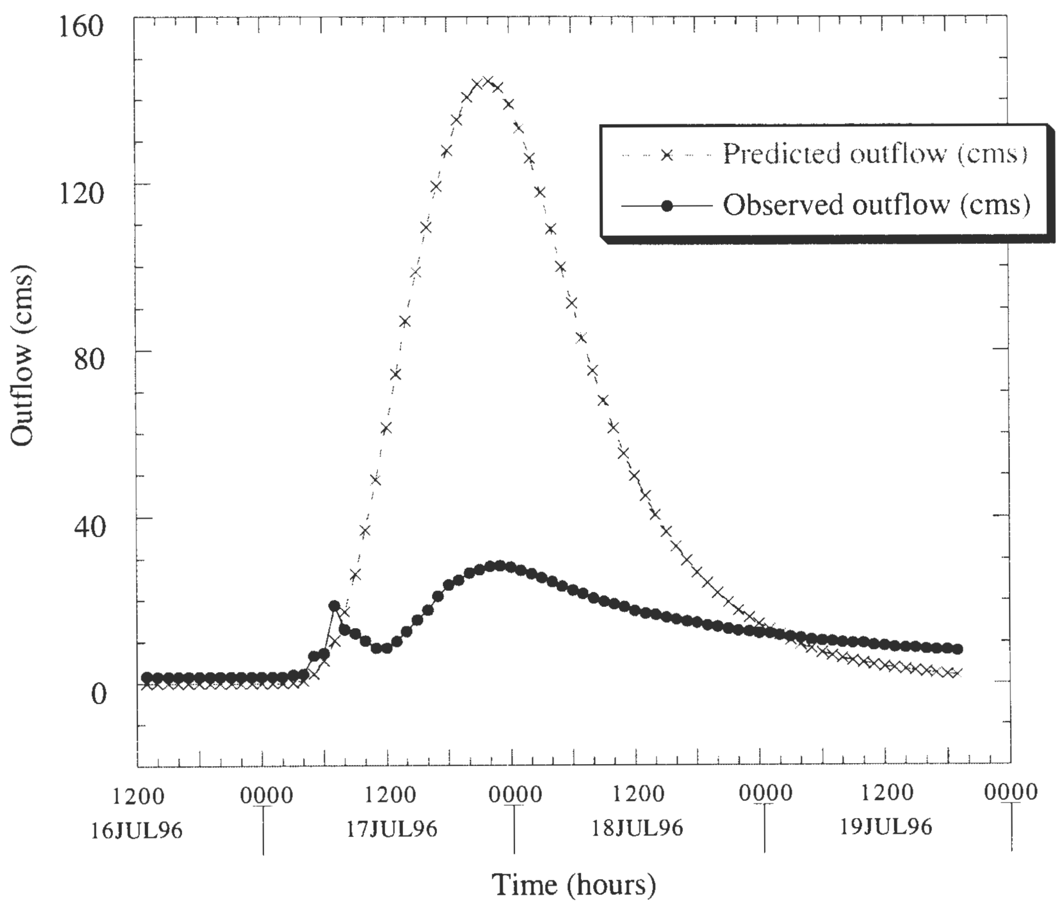

Figure 7.3. Simulation for the July 1996 event using raingage data as input (AMC I)

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7.3.2.1.4. July 1993 flood event:

The Flood of 1993's intensity was historically unusual. The Water Year of 1993 was the wettest in 121 years of record (Snyder & Associates Inc, 1996). Rainfall totals from January to July were one-half to two times the average precipitation for the same period (Parrett et al., 1993). In addition, the snow cover of the 1992/1993 winter was heavy, and its melting in spring contributed to soil saturation (Snyder & Associates Inc., 1996). By spring, soil saturation was at 85%, which meant a larger percentage of new rainfall occurring was to become runoff. Using loss rate parameters corresponding to AMC III was thus perfectly justified.

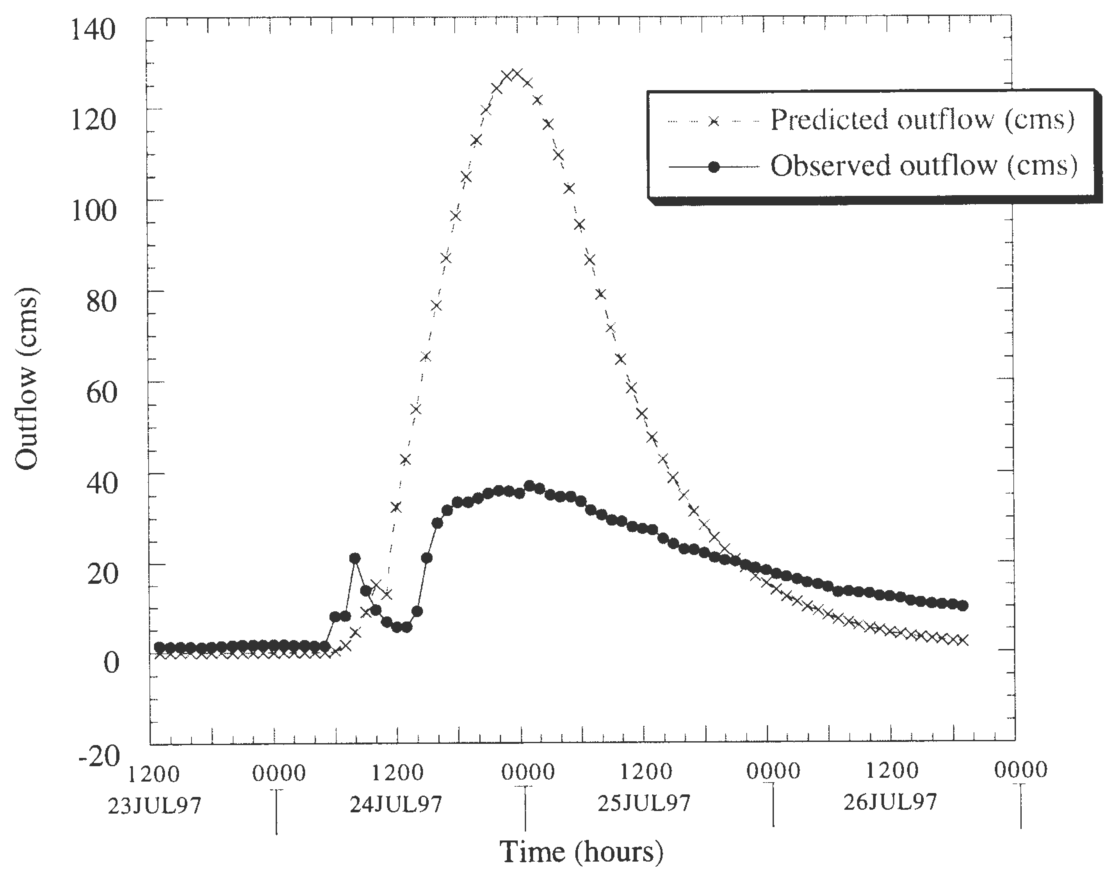

Figure 7.4. Simulation for the July 1997 event using raingage data as input

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Overall, results from the simulations showed a satisfactory matching between the computed hydrographs and the observed ones (Figure 7.5). Concerning the record crest of Squaw Creek on July 9, 1993, which caused a tremedous extent of building damage in Ames, the peak flow was predicted to occur at 03:00 PM with a magnitude of 601.82 cms, corresponding to a river stage of 5.47 m (the flood stage is 2.1 m) (Appendix C). In actuality, the flood wave hit Ames at 09:00 AM that same day, with a magnitude of 671.11 cms, corresponding to a river stage of 5.62 m. The flow prediction was thus close to reality in terms of its dimension, but predicted later. The timing of the computed second peak is right on time, on July 11, at 01:00 PM, but somewhat overpredicted with a magnitude of 379.43 cms (corresponding to a river

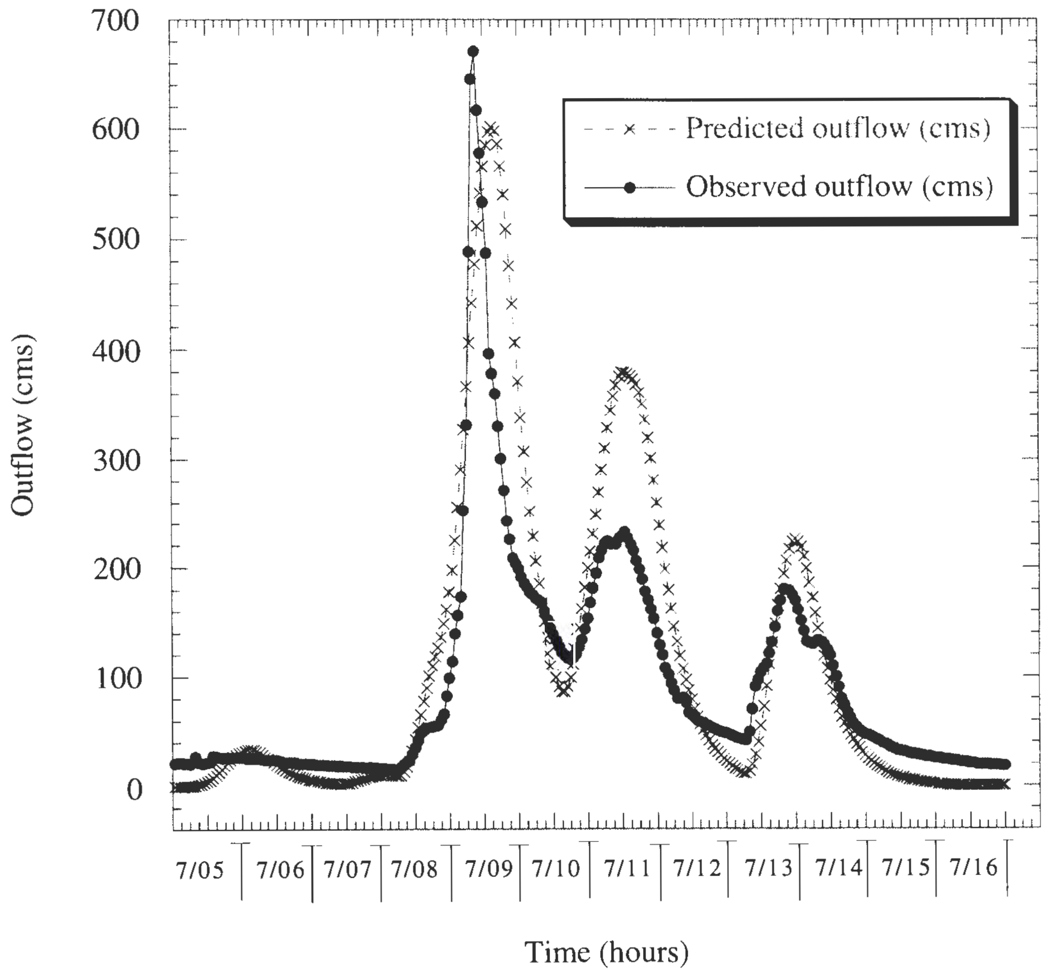

Figure 7.5. Simulation for the July 1993 flood using raingage data as input

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stage of 4.83 m) versus an observed value of 232.20 cms (corresponding to a river stage of 4.16 m). As far as the third peak on July 13 is concerned, the prediction is for a discharge of 225.41 cms (corresponding to a river stage of 4.11 m) at midnight, whereas an actual peak of 180.38 cms (corresponding to a river stage of 3.76 m) occurred at 08:00 PM. This forecasting was thus also somewhat late and overpredicted.

7.3.2.2. Quasi-distributed ModClark model

7.3.2.2.1. June 1996 flood event:

The ModClark simulation for the June 1996 event used the same subbasin loss rates as the run using raingage data: AMC III. Results from simulations with HMS predicted a peak flow of 53.80 cms on June 17th at 08:00 AM (Figure 7.6). Comparison with observed flow data, a peak of 359.62 cms occurring on June 17th at 10:00 AM, indicated a large underprediction of about 85% in magnitude. This predicted flow's corresponding river stage (1.86 m) was not above the flooding stage of 2.1 m (Appendix C). A flood warning for possibility of damage to life and property would not have been issued based on these results. As far as the prediction of the timing of the peak is concerned, it was close to the one of the actual peak, though a little earlier.

Comparing the results of the ModClark simulation using radar data with the one of the Clark simulation using raingage data indicated that the Clark gage model performed better, with a peak timing and magnitude closer to the observed event data (Figure 7.6).

7.3.2.2.2. July 1996 high water event:

ModClark simulations for the July 1996 event used the same subbasin loss rates as the runs using raingage data. Two different soil moisture conditions were used: AMC II, and AMC I. Simulations with HMS for AMC II showed a peak flow of 83.66 cms on July 18th at Noon (Figure 7.7). The magnitude of the predicted flow was higher than the one of the observed flow, 28.32 cms. Compared to the observed time of the peak discharge on July 17th at 10:00 PM, the forecasted peak outflow was delayed by more than 12 hours. The use of AMC I modified only the magnitude of the predicted flow: 15.53 cms, which corresponded to an underprediction (Figure 7.8). These results indicated that the most adequate CN would have been somewhere between AMC II and I, for

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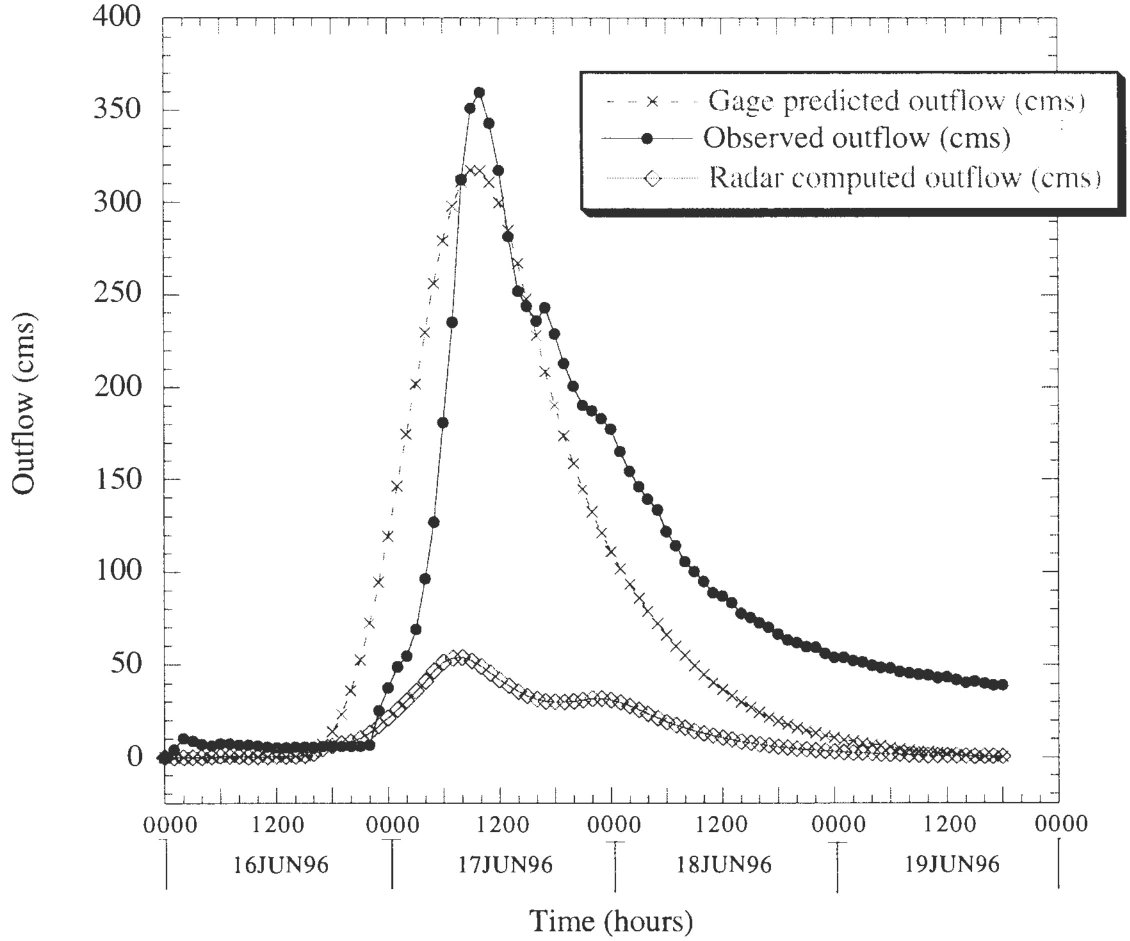

Figure 7.6. ModClark simulation for the June 1996 flood using NEXRAD data as input

the predicted magnitude to exactly model the observed one. Using AMC I.5 gave a predicted discharge of 43.13 cms at Noon, which corresponds to a slight overprediction (Figure 7.9).

Comparison of these ModClark simulation results using radar data with those of the Clark simulations using raingage data led to two observations (Figures 7.7, 7.8, and 7.9). The Clark model was correct for the peak time, but greatly overpredicted in two cases. The ModClark model was close in terms of magnitude but wrong for the timing. Neither prediction was best.

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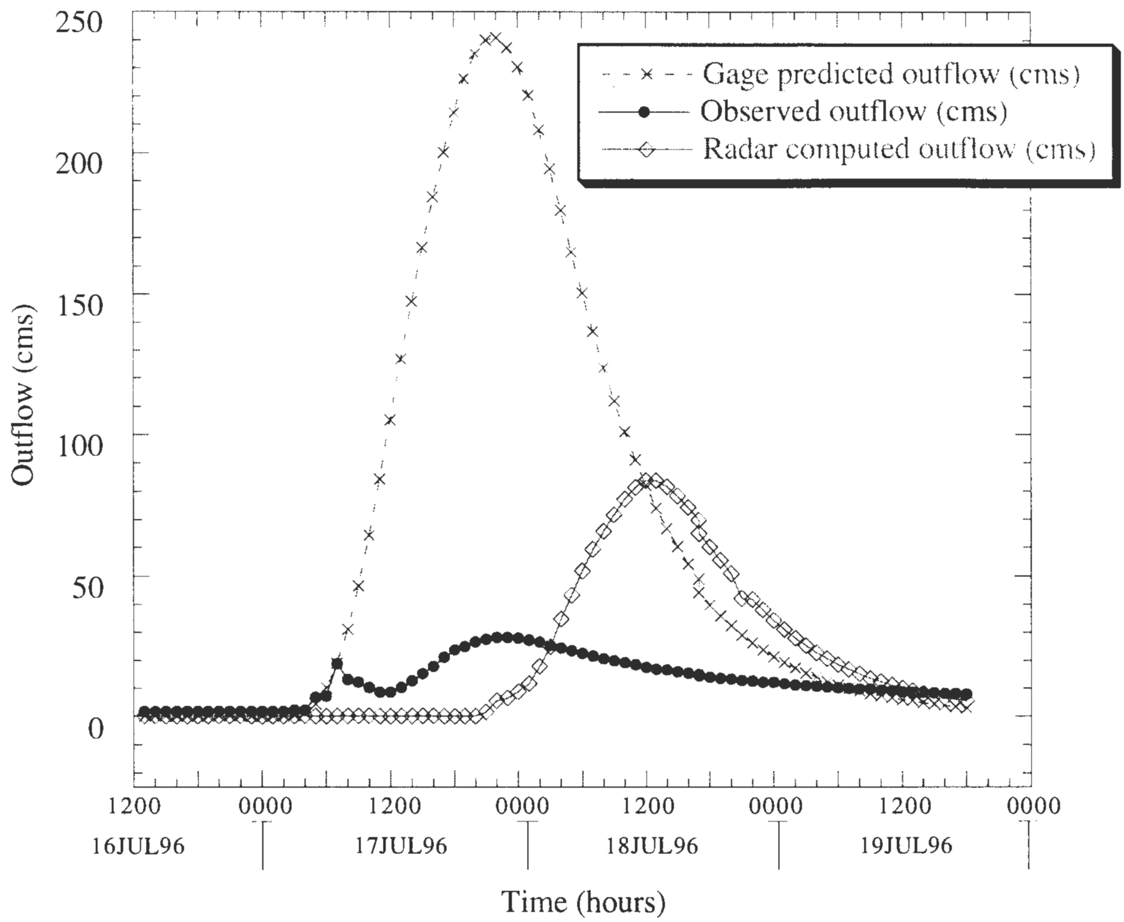

Figure 7.7. ModClark simulation for the July 1996 event using NEXRAD data as input (AMC II)

7.3.2.2.3. July 1997 high water event:

ModClark simulations for the July 1997 event used the same subbasin loss rates as the runs using raingage data: AMC I. Results from simulations with HMS indicated a predicted peak outflow practically equal to zero (Figure 7.10). This did not compare with the actual observed flow of 37.01 cms. The timing of the prediction, when compared to the actual time of the peak discharge, showed the computed peak was 10 hours early: 24 July 97 at 02:00 PM versus 24 July 97 at midnight for the observed. HMS subbasin data results indicating a very small amount of rainfall input, another simulation was done, using AMC II instead. The forecasted peak outflow was then 1.77 cms on July 24, at 03:00 PM (Figure 7.11). The timing was about the same but this run pointed out the small quantity of precipitation detected by NEXRAD.

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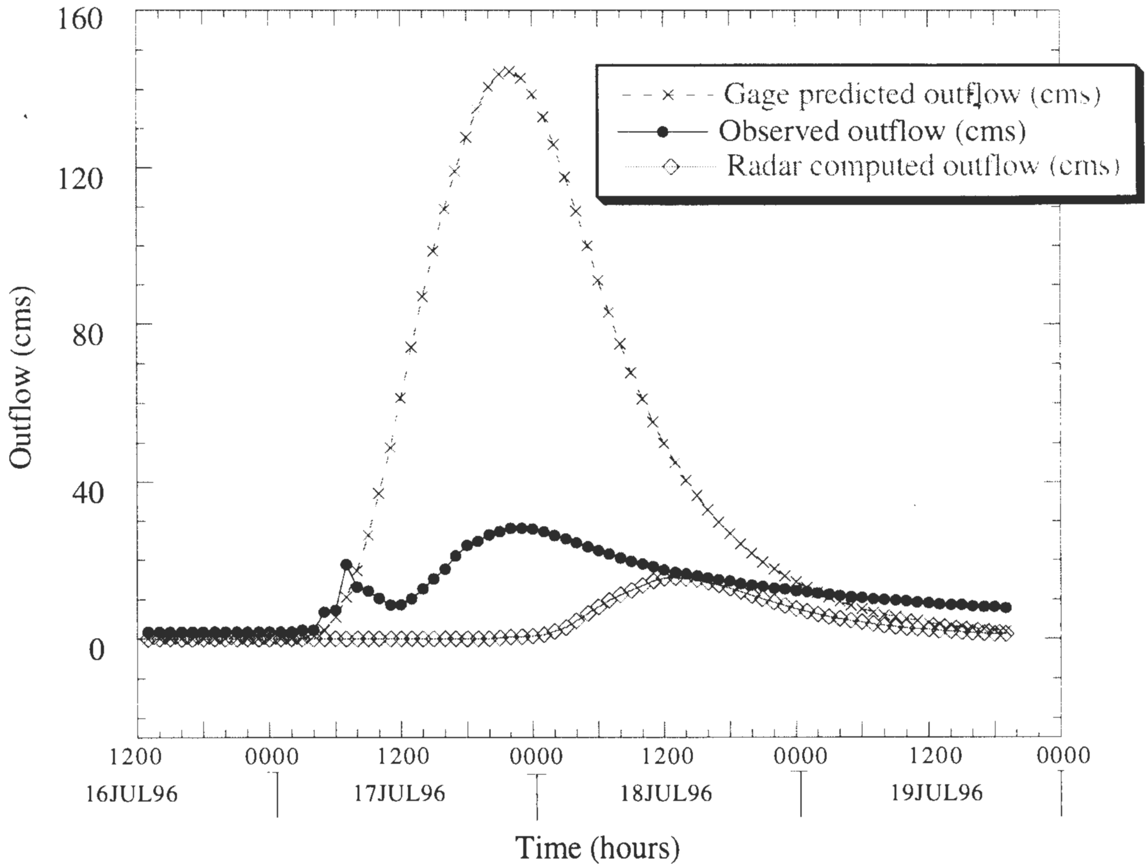

Figure 7.8. ModClark simulation for the July 1996 event using NEXRAD data as input (AMC I)

Comparison of the ModClark simulation results using radar data with those of the Clark simulations using raingage data showed an overprediction of outflow in one case (Clark), and an underprediction in the other (ModClark) (Figures 7.10 and 7.11). Like with the July 96 runs, the timing of the peak was correct with the Clark model results, but it was early with the ModClark model results. Neither prediction was best.

7.3.2.2.4. Analysis of radar data and results

Radar data simulations giving some unexpected results, with large errors in runoff hydrograph magnitude and peak time, and the GridLoadhdp program being fairly new and still in its developmental stage, some thorough examination of the NEXRAD data itself appeared necessary.

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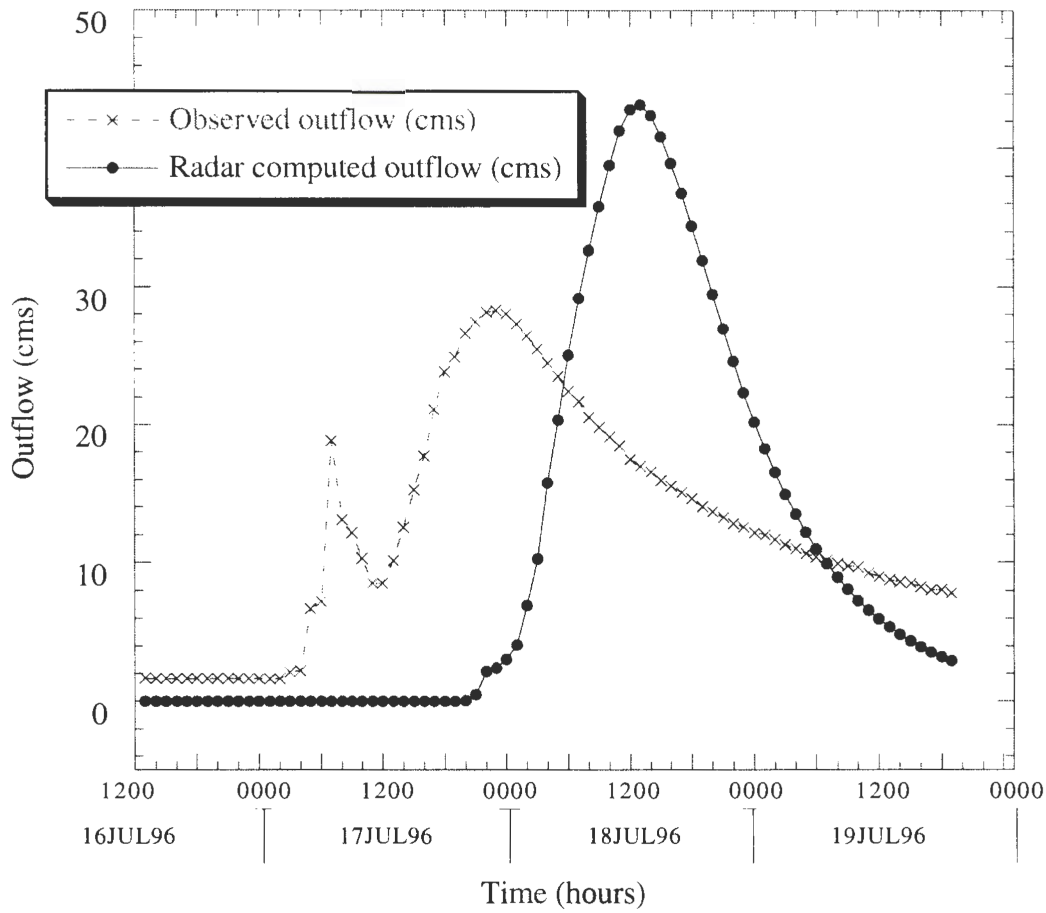

Figure 7.9. ModClark simulation for the July 1996 event using NEXRAD data as input (AMC I.5)

Original runs with the processed data, in particular, gave no output at all for the July 1997 case. The GridLoadhdp program developed by HEC is a brand new unpublished program, which has only been used in very few occasions for the application of the ModClark model by HEC. During the collaboration work, HEC agreed to check on its program since the Squaw Creek results seemed erroneous. A mistake was found in the program, which was causing rainfall values to be reduced by a magnitude of the order of 10. Re-run of the three simulations then led to the results described previously.

To make sure that the radar technical specifications input to Gridloadhdp were correct, images of level I NEXRAD data for the basin were obtained for the three events -- in the format of printouts or .gif files -- from the contact

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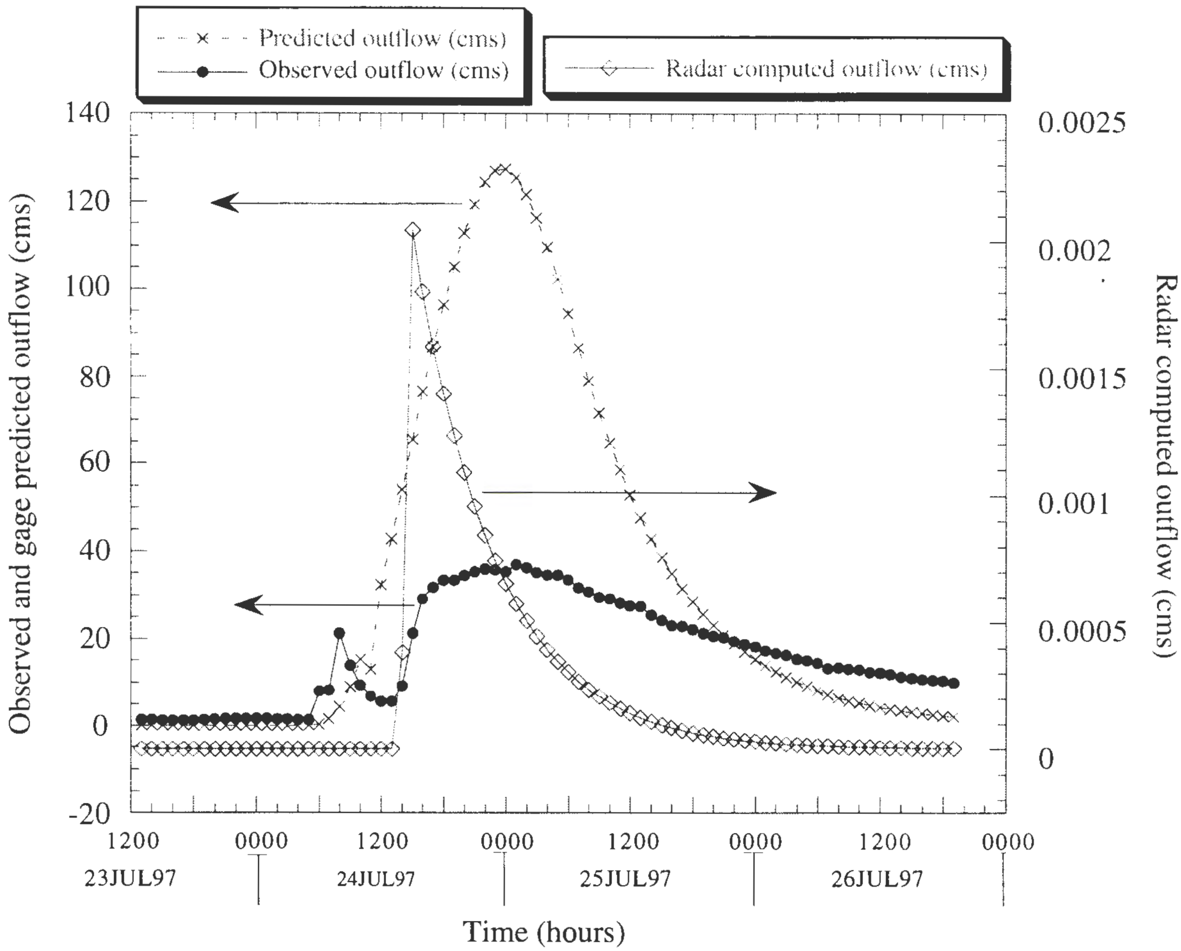

Figure 7.10. ModClark simulation for the July 1997 event using NEXRAD data as input (AMC I)

person at the NWS in Johnston, Iowa. Some data could not be retrieved for the July 1997 storm, but a few of the storm hours were available. GridLoadhdp automatically generates, after translation of the data into a DSS format, .jpg image files for each hour of the data. These images were then compared to the ones of the NWS. For the three events, it was found that both sets of images were agreeing. The storms were moving in the same direction and, except for some hours, precipitation was present on both sets for the same periods.

A more in-depth examination of the radar data was then pursued. The HEC GridLoadhdp person specially generated a report data file for each month of the event, reporting for each hour and for the whole radar beam, the number of HRAP cells receiving more than 0, 10, and 20 mm of precipitation, as well as the maximum amount of precipitation detected. These data were

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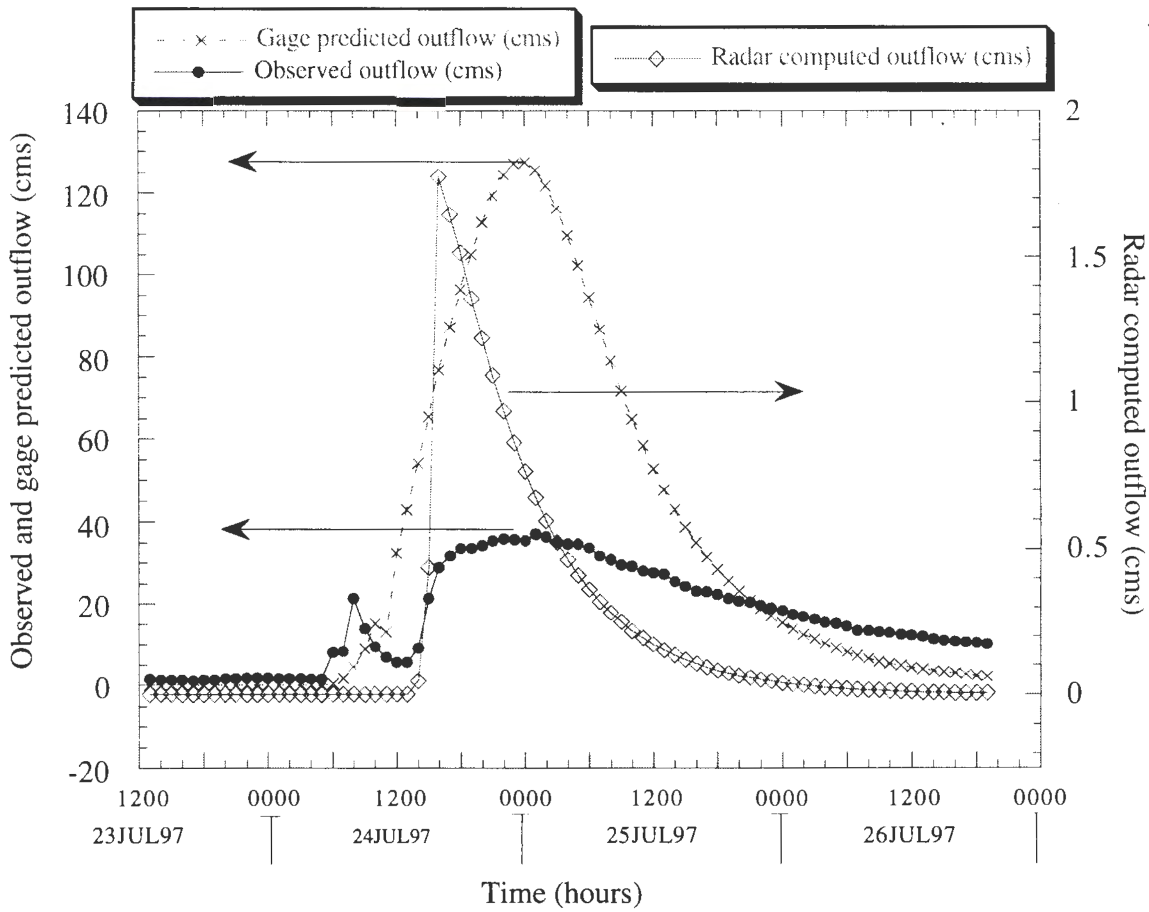

Figure 7.11. ModClark simulation for the July 1997 event using NEXRAD data as input (AMC II)

analyzed in comparison with the timing of the precipitation received by the raingages. because this information was concerning the whole radar beam, it was not easy to determine which cells' data would apply to the Squaw Creek watershed. The Gridloadhdp program was then slightly altered so as to include in its output a detailed report showing for each basin cell the amount of precipitation received for each hour. This was done using the HRAP coordinates of the cells covering the basin that were obtained from the NWS GridParm procedure. These files' data were then analyzed and, after conversion of the storm times from UTC to local time, compared with the timing of the precipitation received by the NWS raingages and with the NWS images for the Squaw Creek area. For the June 1996 storm, the timing of the

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rainfall agreed fairly well in all cases. For the July 1996 event, the radar did not seem to have captured the storm at the same time as the raingages did: an approximate delay of 4 to 5 hours was noticeable. The NWS images also indicated a different storm beginning time than the corresponding digital radar data: a discrepancy of 2 to 3 hours was identified. As far as the July 1997 case was concerned, it was obvious that the raingages captured the storm in a totally different way from the radar: both systems recorded about the same beginning of the storm but the raingages showed rainfall data for an extra 6 hours.

In terms of rainfall intensities, a general comparison was made between the raingage data and the radar data. It was found that for the June 96 event, radar rainfall intensities were very much lower than the ones recorded by the raingages. The precipitation intensities were, however, of the same order of magnitude for the July 96 case, with a slight overestimation of the radar values. The July 97 case was characterized by low radar rainfall values, very much lower than the raingage ones.

These observations on the radar data were useful for several reasons. They first confirmed that the GridLoadhdp program had operated well. They then highlighted two facts: 1) the radar rainfall intensities did not always seem to reflect reality, and 2) the radar precipitation did not appear to always be detected at the right times. During the examination of the digital data, it was also found that a few hours of data were missing. The occurrence of a mismatch between digital data and NWS images was intriguing, and may be the consequence of a problem of data transfer between the raw data at the radar location and its capture into the image software and the archival system. It thus appeared that the quality of NEXRAD data may not be the best as of now.

7.3.2.2.5. Testing of the functioning of HEC-HMS

HMS being a brand-new software with several malfunctioning problems already identified, it seemed prudent to test its functioning in the quasi-distributed mode -- ModClark. It was also one of the very first times that ModClark was used in the HMS environment, so checking would ensure that the software functioning was not causing the erroneous hydrographs in the predictions.

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To do so, several different fictitious DSS radar data files were generated using different scripts added to the GridLoadhdp program by the HEC specialist. They were generated so that each hour of the storm would correspond to a uniform rainfall intensity. All basin HRAP cells would receive this same amount of precipitation simultaneously. The purpose was then to use these fictitious radar rainfall data files with modClark in HMS to examine how the model would respond. If the software was functioning well, it was expected to obtain even and uniform runoff hydrographs since a uniform rainfall was used for input.

First, a series of four fictitious storms were utilized: a 1-hour storm with a rainfall intensity of 25 mm/hr, a 1-hour storm with a rainfall intensity of 125 mm/hr, a 3-hour storm with a rainfall intensity of 25 mm/hr, and a 3-hour storm with a rainfall intensity of 125 mm/hr. Running of these DSS radar files was done with HMS ModClark using the different AMC conditions, I, II, and III, so as to examine the model response under different circumstances. The following results were obtained (Table 7.1).

The shape of the runoff hydrographs obtained in all cases was a uniform bell shaped hydrograph, which proved that HMS ModClark was responding well to a uniform rain input. In addition to this characteristic, the magnitude

Table 7.1. HMS ModClark simulation results for the four fictitious storms: peak magnitude and date and time of peak (in hours)

Storm	1-hour 25 mm	1-hour 125 mm	3-hour 25 mm	3-hour 125 mm
Occurrence of storm	7/2, 18:00 to 19:00	7/2, 18:00 to 19:00	7/3, 01:00 to 04:00	7/3, 01:00 to 04:00
AMC I	0 cms	246.83 cms	68.12 cms	1624.70 cms
	-	7/3, 11:00	7/3, 20:00	7/3, 19:00
AMC II	12.35 cms	483.81 cms	204.13 cms	2087.70 cms
	7/3, 11:00	7/3, 11:00	7/3, 20:00	7/3, 19:00
AMC III	62.25 cms	686.30 cms	354.27 cms	2367.20 cms
	7/3, 11:00	7/3, 11:00	7/3, 19:00	7/3, 19:00

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of the outflows computed for the different soil moisture conditions fitted what one would expect: gradually more runoff as the soil water content increases. These run results thus demonstrated that the ModClark model is functioning properly in the HMS environment.

A general outlook at the time to peak in all storm cases revealed an average of 15 to 16 hours. The time to peak here was defined as the time from the center of mass of the rainfall to the peak time of the hydrograph at the basin outlet. Previous studies on the Squaw Creek have found that this time to peak is generally between 12 and 14 hours, though the direction in which the storm travels can influence it (Ganville, 1987; Snyder & Associates Inc., 1996). Such a time being 15 or 16 hours, the hydrograph peak in these fictitious storms appeared somewhat late, though its value, within a reasonable order of magnitude, did confirm a correct functioning of the Squaw Creek ModClark model.

The time to peak being more accurately measured from the time of maximum rainfall intensity within the storm duration to the peak time at the basin outlet, two other fictitious storms were made up that would ease such determination. Using additional scripts added to the Gridloadhdp program two fictitious DSS radar data files were generated that corresponded to the following storms: a 3-hour storm with successive rainfall intensities of 25 mm/hr, 125 mm/hr, and 50 mm/hr; and a 9-hour storm with successive rainfall intensities of 25 mm/hr, 50 mm/hr, 75 mm/hr, 100 mm/hr, 125 mm/hr, 100 mm/hr, 75 mm/hr, 50 mm/hr, and 25 mm/hr. the characteritics of these storms were the same as the previous ones: the rainfall intensity was the same for the hour it applied to, and all basin HRAP cells received this amount of precipitation. These files were used as input to the HMS ModClark model for the three AMC conditions. Results are listed in Table 7.2 and Table 7.3.

Like with the precedent fictitious storms, the same result trends could be observed: a uniform shape of the outflow hydrograph and an increase in computed runoff as curve numbers would increase. This confirmed the observation that HMS ModClark was running properly. These simulations for the two storms also allowed an easy determination of the time to peak. The results showed that the modeled time to peak was longer than 12 to 14 hours.

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Table 7.2. HMS ModClark simulation results for the two additional fictitious storms: peak magnitude and date and time of peak (in hours)

Storm	3-hour	9-hour
Occurrence of storm	7/3, 01:00 to 04:00	7/3, 01:00 to 10:00
AMC I	607.15 cms	3126.30 cms
	7/3/ 19:00	7/3, 23:00
AMC II	945.45 cms	3641.90 cms
	7/3, 19:00	7/3, 22:00
AMC III	1187.30 cms	3921.50 cms
	7/3, 19:00	7/3, 22:00

Table 7.3. Time to peak (in hours) for the two fictitious storms

Storm	3-hour	9-hour
AMC I	17	18
AMC II	17	17
AMC III	17	17

This indicated that, based on this hypothetical radar rainfall data, the Squaw Creek ModClark model probably required some adjustment of its parameters related to the time of travel of the flow because the runoff was reaching the outlet late. Two deductions could be made from these results. Model routing parameters, in particular the Muskingum time of travel of the water wave, would probably need to be adjusted. The model travel times for the HRAP cells -- from the center of the cell to the subbasin outlet -- would also possibly benefit from some adjustment. Adjusting both types of parameters would probably help reduce the basin time to peak.

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7.4. Adjustment of parameters

7.4.1. Lumped Clark model

HEC-HMS has capabilities for an automated optimization of the values of selected runoff parameters when observed precipitation and discharge data are available (Hydrologic Engineering Center, 1998). Parameter calibration is achieved by an automated adjustment of the values of the selected parameters to produce an optimal fit of a computed hydrograph to an observed hydrograph at a target location. In HMS, the optimal fit is quantitatively measured with an objective function based on the degree of variation between the computed and observed hydrographs. The variation is equal to zero if the hydrographs match exactly. The automated parameter estimation is performed with a search procedure which adjusts the selected parameters to produce an optimal fit that minimizes the magnitude of the objective function. Constraints are imposed on parameter values to insure that unreasonable values are not utilized. Initial values for all parameters are required at the start of optimization. A hydrograph is computed at the target location and the value of the objective function is calculated. The search procedure adjusts values for the selected parameters to optimize and a new computed hydrograph and objective function are obtained. This procedure is repeated until little improvement in the objective function is gained.

Calibration for the Squaw Creek model was performed with parameters for subbasins and reaches upstream of the basin outlet in Ames, where an observed streamflow hydrograph can be known. Of the four objective function types available in HMS (HEC-1 objective function, sum of squared residuals, sum of absolute residuals, and percent error in peak flow), the HEC-1 objective function was chosen because of its more accurate measure of goodness of fit. The equation and definition of the function can be found in the HEC-HMS manual (Hydrologic Engineering Center, 1998). Two search methods are available in HMS: the Univariate Gradient Method, and the Nelder and Mead method. The latter method was chosen because of its potential for producing a more optimal fit through changes of the magnitude of all selected parameters during each optimization run. Details on this search method can be found in the HEC-HMS manual and in a journal article (Nelder and Mead, 1965).

Values for the constraints imposed on the parameters are divided into two groups (Hydrologic Engineering Center, 1998). Hard constraints are those that

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keep the magnitude of a variable within reasonable limits. For example, negative values are not allowed. Values for hard constraints are indicated in Table 7.4.

These constraints are general guidelines. Soft constraints need to be specified by the user to keep parameter values within tighter limits than those defined by the hard constraints. They also help avoid values that cause instabilities or errors in computations.

Table 7.4. Hard constraints on the magnitude of some parameter values as recommended by HEC (Hydrologic Engineering Center, 1998)

Parameter	Minimum constraint	Maximum constraint
Clark time of concentration	0.1 hr	500 hr
Clark storage coefficient	0 hr	150 hr
Muskingum K	0.1 hr	150 hr
Muskingum x	0	0.6

Parameter calibration was performed with HMS only for the lumped Clark model. Simulation results with the ModClark model were not judged satisfactory enough to warrant calibration in this manner. Model transform and routing parameters (Clark time of concentration T_c, Clark storage coefficient R, Muskingum flood wave travel time K, and Muskingum weighting factor x) were used in the optimization process. These parameters depict the watershed's physical characteristics and are held constant in the model. The best possible estimate of their value is consequently critical for the reliability of the model. Parameters such as the soil moisture, represented by the curve number, and the baseflow, vary from storm to storm: they are thus not part of an optimization process.

The Clark model was calibrated with the June 16-17 1996 event -- the best of the three simulations --, using observed flow at the outlet and observed raingage data for the hydrograph computation. Original values of the parameters served as the initial values in the optimization. After numerous trial runs, soft constraints were defined so as to prevent instability in the computation of calibrated values, and obtain reasonably calibrated

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parameters. The best soft constraints leading to zero warnings and no errors in the optimization and to the best obtainable fit are reported in Table 7.5.

The optimization process was tedious, mainly because of the difficulty in determining values for the soft constraints. It was found that optimizing over a long time period, for example 16 June 96, 01:00 to 19 June 96, 18:00, did not produce satisfactory results at all in terms of improving the peak flow and its magnitude, though it improved the overall shape of the hydrograph. Reducing the time window a lot, on the other hand, would render the procedure easier and faster, but exclude a large portion of the hydrograph. An intermediate time frame. encompassing most of the rising and falling limbs of the hydrograph, was thus chosen for the computations: 16 June 96, 12:00 to 18 June 96, 18:00. The purpose was to try and get a reasonable calibration for parameters within this time window because the interest is to model the peak flow and time as well as possible so, if it is used in the future by the City of Ames, their flood warning system can be as reliable as possible.

Table 7.5. Soft constraints on the magnitude of parameter values used in the optimization process

Parameter	Minimum constraint	Maximum constraint
Clark time of concentration	8 - 9 hr	20 hr
Clark storage coefficient	1 hr	20 hr
Muskingum K	1 hr	6 - 10 hr
Muskingum x	0.1	0.3

Once optimization results were judged satisfactory, that is to say when the value of the HEC-1 objective function showed little or no improvement , and the values for the parameters were within reasonable ranges and not unrealistic, the optimized values were retained. They correspond to a value of 36.4 for the objective function. Overall, optimized values were not too different from the original estimated values. They are listed in Table 7.6.

These optimized values, when used to re-run the June 96 simulation, helped to obtain a better fit between the observed hydrograph and the computed one, on both the rising and falling limbs. Matching between the computed and

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Table 7.6. Lumped Clark model optimized parameter values

Parameter	Optimized value
Subbasin-A Tc	9.00 hrs
Subbasin-A R	9.56 hrs
Subbasin-B Tc	13.09 hrs
Subbasin-B R	11.72 hrs
Subbasin-C Tc	10.01 hrs
Subbasin-C R	9.77 hrs
Reach-1 K	6.13 hrs
Reach-2 K	5.99 hrs
Reach-1 x	0.11
Reach-2 x	0.30

the observed falling limbs appeared to require more adjusting. The new simulation showed a peak of 329.19 cms, occurring at 11:00 AM on June 17, 96, corresponding to a river stage of 4.64 m (Figure 7.12). Compared to the original results, the calibrated parameters led to an increased peak flow, 329.19 cms versus 317.14 cms. The timing however did not improve and was still off from the actual time by 1hour.

7.4.2. ModClark model

The testing of the functioning of HEC-HMS having indicated the need for adjusting cell travel times and the time of travel of the flood wave, some adjustments were made to the ModClark model. They used the hypothetical radar data storms generated from the testing of HMS. The goal was to modify values so the time to peak would be within the 12 to 14 hour range, which corresponds to the range of time response of the watershed as known to date. The late timing observed during the run of the two storms was small so the adjustment of the parameters had to be progressive.

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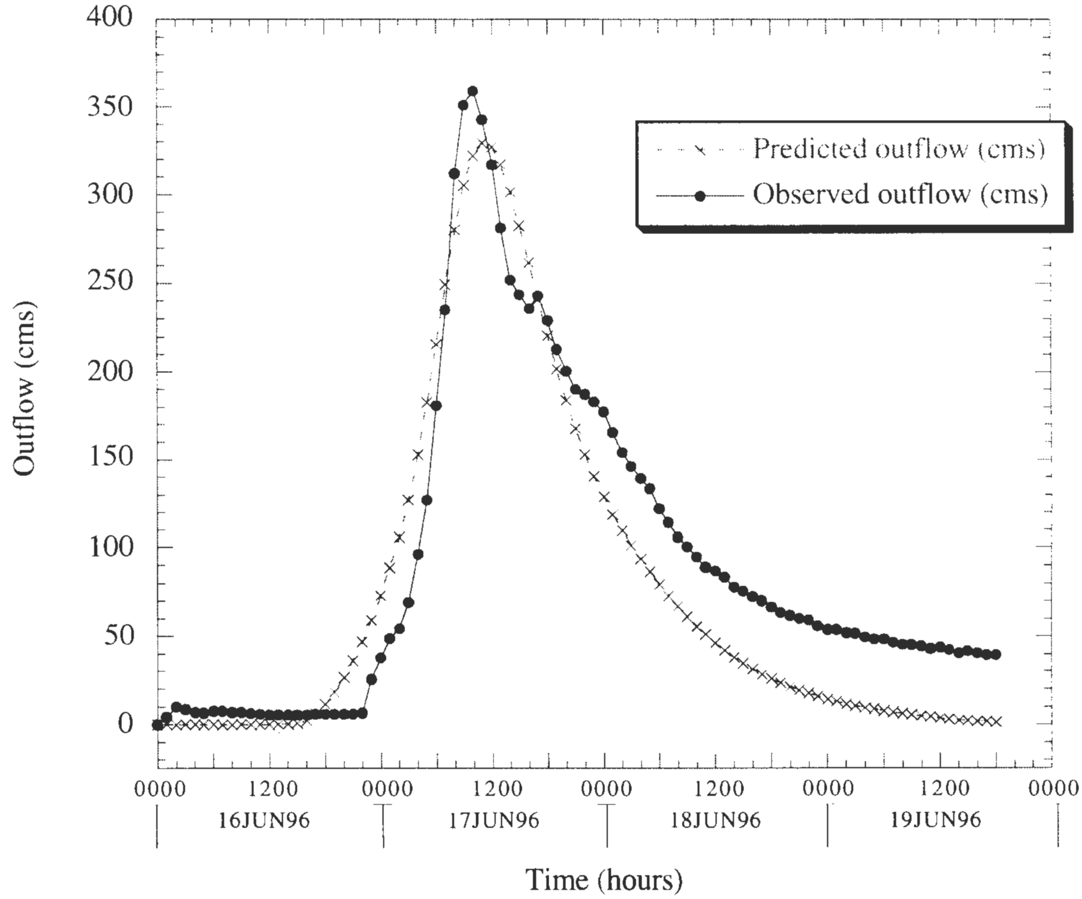

Figure 7.12. Re-run of the simulation for the June 1996 flood using the calibrated parameters

In the first place, the Muskingum K values for the two model reaches were reduced by one hour. Reach-1 value became 3.4 hours and Reach-2 3.3 hours.

The simulations for the 3-hour and 9-hour hypothetical storms were re-run to analyze the effects of the change. Peak time results are reported in Table 7.7.

The time to peak decreased by one hour for the 3-hour storm: 16 hours versus 17 hours originally, and by 2 hours for the 9-hour storm: 16 hours versus 18 hours originally.

More than 12 to 14 hours of time to peak being obtained, more adjusting was needed. The Gridcell parameter file, generated through the gridParm procedure, was then manually edited so the individual cell travel time could be modified. One needs to recall that in the ModClark model, the cells' travel

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Table 7.7. Hypothetical storm simulation results after adjustment of the Muskingum K parameters in the ModClark model

	3-hour storm		9-hour storm	
	Ouflow (cms)	Peak time (hours)	Ouflow (cms)	Peak time (hours)
AMC I	643.11	18:00	3303.10	21:00
AMC II	999.64	18:00	3838.90	21:00
AMC III	1253.10	18:00	4131.50	20:00

time is represented by the flow path length. Observations during the testing of HMS having shown that these times of travel were probably slightly overestimated, the values were all reduced by 1 km. Re-run of the simulations for the 3-hour and 9-hour hypothetical storms gave the following results (Table 7.8).

The impact of the modification on the cell travel time indexes was not too noticeable: only a decrease by 1 hour was observed in two of the cases. The reduction of the cell travel times did not appear to have an obvious effect. It could then be inferred that their estimation through the GIS GridParm procedure was probably correct. The values of the routing parameter K, being most likely overestimated, were probably more in need for some additional adjustment. The K values were thus reduced by another hour: Reach-1's value became 2.4 hours and Reach-2 2.3 hours. The output of the re-run of the simulations for the 3-hour and the 9- hour hypothetical storms is reported in Table 7.9.

This time, simulation results showed a time to peak of 14 hours for both the 3-hour and the 9-hour storms. This value fell within the range of 12 to 14 hours for the basin. These parameter adjustments of the ModClark model were judged satisfactory and should probably be retained for future work on the model. The adjustment work was not pursued any further because the data used was fictitious and real data needs to be used to ensure the validity of

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Table 7.8. Hypothetical storm simulation results after adjustment of the cell travel time indexes in the ModClark model

	3-hour storm		9-hour storm	
	Ouflow (cms)	Peak time (hours)	Ouflow (cms)	Peak time (hours)
AMC I	636.70	18:00	3274.70	21:00
AMC II	989.31	18:00	3809.10	20:00
AMC III	1241.90	17:00	4103.70	20:00

Table 7.9. Hypothetical storm simulation results after a second adjustment of the Muskingum K parameters in the ModClark model

	3-hour storm		9-hour storm	
	Ouflow (cms)	Peak time (hours)	Ouflow (cms)	Peak time (hours)
AMC I	670.55	16:00	3450.60	19:00
AMC II	1043.80	16:00	4014.30	19:00
AMC III	1241.90	17:00	4312.00	19:00

these adjustments. As with the lumped Clark model, it can be called a "first-generation" adjustment.

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CHAPTER 8. DISCUSSION

8.1. Lumped Clark model results

8.1.1. June 1996 flood event

Simulation results for the June 1996 flood in Ames were satisfactory, both in terms of peak outflow and time. The simulation led to a flooding prediction, which is what happened. Although close to the City of Ames' HEC-1 model prediction, it did not use loss rates corresponding to AMC I.5 like the City did, but curve numbers representing an AMC III or wet soil condition. This may be better suited to real conditions taking into account the amount of spring snow melt and the very wet month of May. As far as the rainfall data recorded by the NWS raingages is concerned, it seemed to have been well captured temporally and spatially.

8.1.2. July 1996 and July 1997 high water events

This might not have been the case of the simulations for the July 1996 and July 1997 events, where a large overprediction was obtained. Only the timing of the peak flow was satisfactory in both cases. This would tend to reflect a correct estimation of the Clark parameters, since they control the time of concentration and greatly influence the timing of the discharge.

Hand calculations of the depth of rainfall falling onto the watershed and the depth of runoff corresponding to the observed flow clearly showed a discrepancy. With about 71 mm of incoming rain and 3 mm of runoff for the July 96 case, and about 66 mm of incoming rain and 9 mm of runoff for the July 97 case, it seemed impossible that so much water would have infiltrated. Several things can therefore be hypothesized, that might account for the errors encountered in the results.

In particular, the raingages' data can be questionned. Three of these raingages are located outside of the watershed. It is possible that a localized storm cloud was stationary above one or more of the raingages during the rainstorm, hence causing a large amount of precipitation to be recorded. For the July 97 event, modelers at the City of Ames used input data from their own

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raingages disseminated within and outside of the watershed. They, too, obtained an overprediction with their HEC-1 model and think some clouds must have been immobile for some time over the raingages (Karla Tebben, personal commumication, 1998). Some processing errors could also have occurred during the transfer of data, from the NWS raingages to the NCDC in North Carolina, and its transcription to the monthly publication.

The overestimation of rainfall by the raingages resulted in a wrong spatial averaging of the precipitation through the Thiessen network and an erroneous input to the model, leading to an overprediction. This is actually an illustration of the main problem associated with raingages: they do not capture the spatial distribution of a rainstorm.

Regarding the observed streamflow data, one needs to keep in mind that the gage failed in June 96. Whether the flow gage was completely repaired and fully operational a month later is not certain and when asked this question several times, the USGS would not answer. The flow peak recorded for the event was minimal and could slightly underestimate the real conditions.

Physical characteristics in the upper portion of the Squaw Creek basin could also have played a role in the overprediction of flow. In the headwaters area, numerous potholes and small wetlands are present, and the cultivated fields contain a lot of ground depressions. The potential for storing water at this level is probably high. During a rainstorm, these surface depressions and wetlands may fill up with water before any surface runoff actually starts occurring. Only when these puddles and water retaining areas are completely filled up will water flow over the surface as excess precipitation. If one can reasonably assume that the amount of depression storage in the headwaters is substantial, then it is highly possible that during a rainstorm on a relatively dry soil, little runoff occurs due to this surface storage system. In addition, the presence of a network of drainage tiles may play a role. Under dry conditions, it takes a while for the underground tile system to fill up: the amount of water retained there does not reach the stream immediately but later, which might account for the fact the observed flow is so low under such circumstances. The July 96 and July 97 cases are characterized by relatively dry conditions before the rainstorm. Applying the above theory would explain why so little excess rainfall was observed. The model does not incorporate this water storage factor, therefore leading to a false overprediction, perfectly correct from a

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mathematical standpoint but wrong from a physical standpoint. On the other hand, during a heavy rainfall event, these water retaining areas fill up but become full and see their capacity exceeded. Runoff then occurs just like in the situation that the model applies, and the effects of surface storage are not noticeable any more. In such cases, the prediction appears to be matching reality pretty well. This is for instance the case of the June 96 flood simulation. A slight underprediction is however noticeable. This could be explained by the following fact. It is possible that the underground tile system, in a heavy storm situation, gets filled up rapidly, flushing extra water in the creek, hence increasing the amount of flow, resulting in a large observed flow, which the model underpredicts. For the model to be more accurate, especially for the storm cases occurring under dry soil conditions, one would need to try and modify it. In particular, the inclusion of a fictitious detention reservoir component might help take into account the specific physical characteristics in the northern part of the basin: the ground depressions and the tile network. Such a detention storage system should be designed so as to detain or slow down the runoff, as in a reservoir, and then release it. Part of the retained water could also be removed from storage by infiltration through a porous bottom, or by evaporation. The required storage volume should be based on an analysis of storm event volumes. This possible further study on the model might, by trying to mimic characteristics unique to the Squaw Creek basin, lead to an improved accuracy of the flood and high water event predictions.

Another thing is that HEC-HMS is a brand-new software, released by HEC in April 1998. Like any other new model, it has not been extensively used and thus tested yet. Some of its "bugs" have been identified and are being taken care of. It is possible that others may not have been spotted for the time being, which may result in erroneous outputs. The high-water events of July 96 and July 97 are not flood situations. Maybe HMS cannot respond well to small storms and these two cases would tend to illustrate this fact. HEC-HMS is still in its infancy.

8.1.3. July 1993 flood event

Simulation results for the July 93 flooding period were interesting for several reasons. First, they came relatively close in terms of magnitude and timing, which helped verify that the model parameters estimated in the first

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place were not too far off their correct value. Second, HEC-HMS, like HEC-1, does not respond well to several close peaks of rainfall, according to HEC (Troy Nicolini, personal communication, 1997). The period modeled here did contain such features but the HMS response did not appear erroneous, which would therefore constitute an unusual finding. It is also possible that the difference in peak timing between the actual and computed flows originates in this incapacity of HMS in modeling several successive storms.

8.2. Quasi-distributed ModClark model

8.2.1. Event simulations

Simulation results for the three study cases of the June 1996, July 1996, and July 1997 storms showed computed runoff hydrographs that were grossly in error compared to observed hydrographs. For the June 96 event, there was not enough radar measured rainfall to model the observed flow. Even reducing loss rates to zero would not have improved the prediction and would have been unrealistic. The corresponding lumped Clark simulation using raingage data produced a better prediction, especially in terms of peak magnitude. This case seemed to illustrate the type of magnitude errors that can occur in Stage I, with, in this particular situation, an underestimation of the precipitation amount.

The July 96 event peak, aside from its wrongly timing, was modeled fairly well by ModClark. A slight overestimation of the actual rainfall, however, seemed to have been recorded. This, again, can be classified as typical of the magnitude of errors in the radar measurements. It was interesting to observe that the storm was falsely recorded by the raingages, although the timing of the peak with the lumped Clark simulation was much more accurate. The NEXRAD data inaccuracies were rather small in terms of magnitude.

The July 97 event displayed the same characteristics: the raingage recorded erroneous amounts of precipitation, whereas the radar did not. Again, the timing of the peak obtained with the lumped Clark simulation was much better than the one with the ModClark run. But the rainfall amounts detected by NEXRAD were quite underestimated because even the use of lower loss rates would not produce a predicted hydrograph even close to the observed one. This, once more, was due to magnitude inaccuracies in the data.

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For both the July 96 and July 97 cases, the raingages misinterpreted the storm event whereas the radar captured the rainfall event better, even though there were problems of timing and magnitude accuracy. This illustrates the potential of NEXRAD in terms of its capacity to detect the spatial distribution of rainfall. Using radar rainfall can thus be an advantage. These two cases give an insight into the fact that the ModClark method can have a significant potential for improving forecasting capability provided it is used with adequately accurate radar rainfall.

One could also notice the following trend from the results. When storms are locally intense (case of the June 96 event), Stage I timing inaccuracies appear to be smaller, whereas less intense storms (July 96 and 97 cases) seem to result in greater timing problems. These timing discrepancies could be attributed to errors in the radar measurement of the precipitation itself. The presence of wind or evaporation below the radar beam can not only significantly affect the quantity of rain that falls onto the ground, but also its timing. The rainfall can then touch the ground either early or late, which causes the radar data to be erroneous in terms of timing. The ModClark computations for the July 96 and July 97 events appear to be cases of this type of measurement problem. Both had discrepancies in the timing of the capture of the storm when compared to NWS images and raingage data and were modeled by ModClark with timing errors. Errors in the radar data can thus greatly impact the accuracy of the modeling results.

One study can be cited here because it relates to the problems encountered with the quality of NEXRAD data in this research work. A NCDC study compared Stage III NEXRAD-estimated storm total precipitation with raingage-measured total precipitation for five storm events in Kansas, south-east Texas, Florida, Louisiana, and South Carolina (National Climatic Data Center Research Customer Service Group, 1996). The five events were chosen due to their extensive and damaging nature. The study revealed a difference between the actual (raingage-measured) and the estimated (NEXRAD-measured) rainfalls. In 80% of the cases, the NEXRAD estimates were too low, sometimes by a factor of 2 or 3. NEXRAD-estimated rainfall fell an average of 73.66 mm below the actual amounts recorded by the raingages. Other comparisons made by the NWS at the Tulsa radar site in Oklahoma and at radar sites in the southern plains also indicated that WSR-88D hourly

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rainfall estimates tend to exhibit significant underestimation (Smith et al., 1996a). The inaccuracies of currently available Stage I and Stage III data thus seem to be common.

The analysis of the radar data and the testing of the functioning of ModClark in the HMS environment were very useful for the interpretation of the results obtained. The testing of HMS revealing that the Squaw Creek ModClark model was operating properly, the erroneous aspect of the results due a model fault could be ruled out. The in-depth examination of the radar data did reveal some flaws. Rainfall was not always detected at the right times and not in intensities reflecting reality. The quality of the data was thus greatly impacting the simulation results and causing errors in them. A model can only produce results that are as accurate as the data entered to the model. At this point in the development of the ModClark capability, it is important to separate the evaluation of the method from that of the radar rainfall product used. Because of this unpredictable performance of stage I data, it is recommended that forecasting water control decisions should not be based solely on Stage I radar rainfall. In the future, superior radar rainfall products should be used, such as Stage III, which will include ground-truthing and other quality enhancements. It is useful to note that as these products become reliable and available, they will be able to be used in the current Squaw Creek ModClark model without any model modification. Specific discussion aspects concerning the quality of NEXRAD data can be found in section 8.2 of the thesis as well as comparisons with ModClark simulation results obtained by HEC in previous applications. These are currently the only ones available in literature for comparison to the Squaw Creek results. The quality of the HEC results compares with the one of this project.

8.2.2. Distributed versus lumped modeling

Although no other research work on such modeling with NEXRAD data could be found, other literature articles gave some interesting insight into comparing rainfall-runoff modeling in a distributed and in a lumped manner.

Different studies have investigated hydrologic model response to precipitation inputs of various spatial resolutions. In their work, Wilson et al. (1979) concluded that ignoring the spatial variability of precipitation input,

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even when the total depth of rainfall is preserved, can have significant influences on the runoff hydrograph. Their findings were for a 67 km^2 basin and two types of precipitation input were used: in the first case, one gage was used to define the input, while in the second, 20 gages were used. Beven and Hornberger (1982) arrived at the similar conclusion that the incorporation of distributed inputs would lead to improved simulated hydrographs.

On the other hand, Obled et al. (1994) used 21 raingages to define the input to 9 subbasins representing a 71 km^2 basin. They presumed that providing distributed inputs to the model would improve simulations. But their semi-distributed representation of the basin produced slightly worse results than a lumped representation. These authors were unable to prove the value of using distributed rainfall inputs to improve hydrologic predictions. Pessoa et al. (1993) found that simulated hydrographs from a 840 km^2 watershed using distributed C-band radar inputs were not significantly different from simulated hydrographs produced from lumped radar-rainfall inputs. Kouwen and Garland (1989) examined the effects of radar data resolution on runoff hydrographs produced from a distributed model. They found that coarser resolution radar input sometimes produced better simulation results, due to smoothing of errors present in finer resolution data. Obled (1991), in a reflection on rainfall data requirements, also concluded that the requirements for the rainfall input are not necessarily of highest resolution. Rather, the choice should come from a consideration of the sensitivity of the model and its overall average performance. Loague and Freeze (1985), who studied the response of a variety of lumped and physically-based models, determined that simpler, less data intensive models provided as good or better predictions, which is food for thought.

In general, there weren't too many articles reporting on the effect of the resolution of the input data on the hydrologic modeling response. NEXRAD being recent, there was also no evaluation of model response to HRAP-based precipitation estimates. Among the studies reported above, there did not seem to be a clear trend supporting the intuitive hypothesis that the use of higher resolution data leads to better model results. One should expect that distributed models, by taking into account the spatial variability of input and processes, would improve flow predictions. but expecting that our improved understanding of these hydrological processes directly affects the quality of the

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model output is probably unjustified. There are many things about hydrological systems that are essentially unknowable, especially the nature of flow processes below the ground (Beven, 1996). Even more computer power cannot remedy this lack of information. Distributed modeling needs to be approached with some circumspection. Some process descriptions may not be appropriate, and some grid parameter values not correctly estimated. There is always a part of uncertainty in distributed models and, in practice, the spatial discretization in the model makes it very difficult, if not impossible, to validate on the field at the grid level. In all cases, at some level in the model, some lumping has to be done (Singh, 1995). Hence, there are no fully distributed models, rather they are quasi-distributed at best. Having both a lumped and a quasi-distributed models for the same basin might represent the best option to insure correct flow predictions.

At this point in time, and in view of the radar data simulation results obtained in the Squaw Creek case and in the previous HEC applications (see section 8.3), it could seem unclear whether the use of gridded rainfall data from NEXRAD with a quasi-distributed model like ModClark can provide more accurate hydrograph simulations than the use of a lumped approach. The joint involvement of radar hydrometeorologic researchers in improving the accuracy of the NEXRAD data, and hydrologic modelers implementing the quasi-distributed model will tell. More work is thus needed because the simulation results obtained only reflect the current stage of the technology.

One thing, however, remains certain for the Squaw Creek case. The presence of a surface water storage system in the northern part of the basin can only be well modeled through a distributed representation. The use of ModClark can therefore only benefit the modeling of the basin and the hydrologic predictions.

8.2.3. Possible weak points of the ModClark model

ModClark model shortcomings as shown in the simulation results for the three study events illustrated the errors that stage I NEXRAD data can contain. Aside from the quality of the input data, the model itself could possibly benefit from certain ameliorations.

In particular, the current structure of ModClark in HMS only allows the specification of a subbasin loss rate that applies to all HRAP cells located in

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this subbasin. Having the possibility of entering grid-based loss rates might help improve flow computations. An extensive soil study for each subbasin would, of course, be required, so as to enable the correct estimation of a loss rate for each grid cell. This might make a difference in the results obtained because soil moisture is a key parameter in making flow predictions. Future versions of HMS will most likely include the capability for a gridded loss rate feature. It will then be interesting to do more research to find out if more spatial resolution for soil mosture will help improve the predictions.

In its future planned features to be added, HMS should incorporate a continuous soil moisture accounting option (Hydrologic Engineering Center, 1998). Soil moisture being a critical factor, one might consider using this new capability in the future for Squaw Creek. In fact, several hydrologic studies with data from various midwestern basins have confirmed that soil moisture is the most important variable for the study of hydrologic processes (Georgakakos et al., 1995). More research would be needed to devise the necesary soil moisture measurement instrumentation, and collect the necessary data. The SCS CN used until now in the Squaw Creek basin remains an empirical method and, if possible, switching to a different loss rate method may bring more confidence to the modeling operations. According to Burnash et al. (1995), in areas where substantial drying of the soil could occur between runoff events, the CN method has difficulty in describing the degree of dryness and determining the amount of runoff produced from rainfall. If storms occur in close proximity to one another, the ability of the technique is degraded. Limitations on the use of the CN method are thus clear.

The ModClark model does not take into account evapotranspiration in its computations. Evapotranspiration includes transpiration from leaf surfaces, direct evaporation from the soil, and direct evaporation from the surfaces of ponds and streams. It is one of the most difficult processes to evaluate in hydrology (Burnash, 1995). It can therefore represent a significant source of error in streamflow simulation. In the case of Squaw Creek, potential evapotranspiration exists in spring and summer, when most flood predictions are made. Surface water storage in the northern part of the watershed contributes to the overall evapotranspiration. Having some means of taking into account the amount of evapotrasnpiration over the whole basin might help improve the accuracy of flow predictions. The same might be true with

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subsurface stormflow (or interflow), which is not part of HMS capabilities. It corresponds to some of the water that, during a storm, infiltrates the soil and moves laterally through the upper soil zones until it enters the stream (Bedient and Huber, 1989). Including this component in the ModClark model might be worth considering.

The modified Clark method, as developed by HEC, is based upon the assumption that flow velocity is constant over a subbasin (see Chapter 4). This assumption allows the flow path length to serve as the cell travel time index in the calculation of the travel time for each subbasin cell. Maidment et al. (1996) have proposed the incorporation of what they call a "spatially distributed velocity field" as an alternative to this assumption. In their theory, the travel velocity through a cell is assumed to be proportional to the cell slope and to the accumulated area of all cells contributing runoff to the cell. Calver (1993) studied ways to ameliorate the time-area runoff calculations. From the recognition that the different parts of a subbasin differ in their response to rainfall, he concluded that taking into account the topographic characteristics of a subbasin is needed to obtain acceptable simulations. In particular, he emphasized a difference in flow velocity throughout the subbasin with a decline with distance from the outlet. Beven et al. (1988) have also pointed out that basin morphology can act as a dominant control on water flow paths and influence hydrologic responses. Better estimations of cell travel times would then be achieved. The possible incorporation of such concept in the ModClark model might be worthy of further study, if a refinement of the method is pursued.

Finally, one last aspect of the ModClark model needs to be pointed out. Flat terrain watersheds like Squaw Creek require a burn-in procedure onto the DEM. This necessitates even more knowledge of Arc/Info compared to what the GridParm user already needs to be familiar with. Because this could represent an obstacle to the development of the use of ModClark in the water resource community, HEC should consider trying to transform these manipulations into macros that could be added to the existing GridParm set of programs. One difficulty inherent to GridParm is the time it takes for each of the processing steps. Adding extra steps would only increase the working time. However, the procedure only needs to be performed once for the study watershed, unless a change in a subbasin outlet location is desired.

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8.3. NEXRAD data

Although the NEXRAD weather radar is becoming a familiar remote sensing technology to the public through Doppler images seen on television and on the Internet, the quality of the information it carries can leave a false impression. One needs to first remember that radars do not measure precipitation directly (Rinehart, 1991). Instead, radars measure the power reflected off objects in the path of the radar signal, including raindrops, snowflakes, and ice. Several potential errors may affect this measurement (James et al., 1993; National Weather Service, unknown date; Smith et al., 1996a). The interception of the radar beam with the ground causes ground clutter and results in an overestimation of the returned power, hence of rainfall. A similar overestimation can be caused by an anomalous propagation of the beam when atmospheric conditions are not standard, increasing the echoes returned. Incorrect hardware calibration can also affect the accuracy of the rainfall estimate. The radar normally corrects the reflectivity value each volume scan so that a proper value is used to derive the rainfall rate. But external or internal noise can cause the reflectivity value to depart significantly from optimum calibration. The presence of mixed precipitation -- rain mixed with hail, snow, or sleet, can produce large reflectivity values, causing an overestimation of rainfall rates. Strong winds below the cloud base can blow rainfall away from the spot on the ground below the portion of the target being sampled. Evaporation below the radar beam can cause the radar to overestimate how much rain is actually falling on the ground. Therefore, it should be clear that the WSR-88D can never estimate rainfall rates with complete accuracy. There are some limitations to the radar because of these factors that cannot be totally controlled by humans.

The need for correction of the rainfall estimates is thus obvious. Several researchers have started looking into this problem and the development of suitable corrective alogorithms and techniques is under way (Anagnostou et al., 1998; Borga and Di Luzio, 1992; Ciach et al., 1997; Crosson et al., 1996; Seo, 1998). At the NWS, an emphasis has been put on quality control of the data. This is included in several precipitation-processing systems providing different levels of refinement in NEXRAD data (Smith et al., 1996a; Appendix B). NEXRAD reflectivity data can be corrected for ground clutter and anomalous propagation of the radar beam using a reflectivity outlier test

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(National Weather Service, unknown date). A radar range correction can be applied to adjust the reflectivity of targets at long ranges, which tend to be reduced. During the precipitation accumulation process, a system checks for missing scans and if too many are missing, no hourly accumulation is performed. Another system can also check for outliers on the hourly accumulations, so as to remove clutter which has passed the previous reflectivity outlier test. Finally, accumulation data could be adjusted based on available raingage data by comparing hourly precipitation from raingages to associated radar values and estimating a mean field bias correction value using a Kalman filter. This would enable the elimination of errors in the NEXRAD data. Currently, the NWS is still working on the development and implementation of the adjustment of radar estimated with raingage data (Karl Jungbluth, personal communication, 1998). There is no correction by raingage data for the entire contiguous United States yet: this will happen in another phase of the NWS modernization process with NEXRAD. The gage adjustment of the data is only done at a few NWS offices at this point in time (Troy Nicolini, personal communication, 1998). On-line raingage networks will be needed for each WSR-88D's site and this is not in place yet.

The quality of currently available NEXRAD data is therefore not the highest, which accounts for the difficulties encountered when modeling with it. Results obtained when using level I data as input to the rainfall-runoff model revealed a large variation in accuracy, a direct consequence of the poor quality of the level I data. Research results from HEC's previous studies at the Tenkiller lake in Oklahoma (Peters and Easton, 1996), the Salt river basin in Missouri (Hydrologic Engineering Center, 1996a), the Muskingum river basin in Ohio (Hydrologic Engineering Center, 1996c), and the Squaw Creek basin do confirm these trends. Sometimes radar estimates are accurate -- case of the Tenkiller lake simulations --, sometimes they aren't. This is reflected in the way the ModClark NEXRAD model performs: either it works well, or it doesn't. Other times, -- case of the Squaw Creek basin -- the lumped Clark gage model performs much better. The discharge peak timing may be early or late, and the peak can be 50% too high or 80% too low. Figure 8.1 shows some of the simulation results obtained in the Muskingum river basin (Hydrologic Engineering Center, 1996c). These results illustrate model shortcomings resulting from magnitude errors contained in Stage I NEXRAD data. In this

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particular case, there was not enough radar-measured rainfall to produce the runoff volume needed to model the observed flow. This situation strongly resembles the one of the June 96 event in the Squaw Creek basin. Figure 8.2 depicts some of the simulation results obtained in the Salt river basin study (Hydrologic Engineering Center, 1996a). There, again, results were not satisfactory. The ModClark computed peak flow was delayed when compared to the actual discharge, and did not have the same magnitude. This output is comparable to those obtained with the July 96 and 97 runs in the Squaw Creek basin. It is also interesting to note that even when using Stage III data for the ModClark simulations -- the Tenkiller case --, some problems were still present: the magnitude of the computed flows was found to greatly depend on a correct estimation of loss rates, which gave uncertainty to the results (Peters and Easton, 1996).

Therefore, the thinking here is that radar rainfall estimates are not developed sufficiently yet for making reliable quantitative forecasts. One needs to wait before using this type of data for reliable water-control decisions, otherwise one will inevitably face difficulty in disentangling the effects of errors in the radar simulations. The use of NEXRAD data in hydrologic modeling at this point in time appears to be somewhat premature: researchers need to wait for improved data quality. The level I data used in the Squaw Creek project seemed to contain a lot of errors in it. Using Stage III in the simulations could not be done due to logistic reasons. The Minnesota RFC indicated that stage III being directly created from Stage I, errors in Stage I would also appear in Stage III. Hence, using Stage III would not have brought any improvement to the current radar simulation results at this point in time. Although central Iowa has no mountains that would have been a cause of clutter, there probably was some kind of clutter within the Stage I data. The humidity of the air, usual in the summer months, might have contributed to the presence of some clutter. The Johnston radar has also been installed a short time ago, so it might be possible that the calibration of the hardware system is not completely mastered yet, and that more experience is to be gained by the NWS radar operators. One needs to keep in mind that the WSR-88D is still a young technology.

The advent of NEXRAD however remains a progress in the field of water resources because of its high spatial and temporal characteristics.

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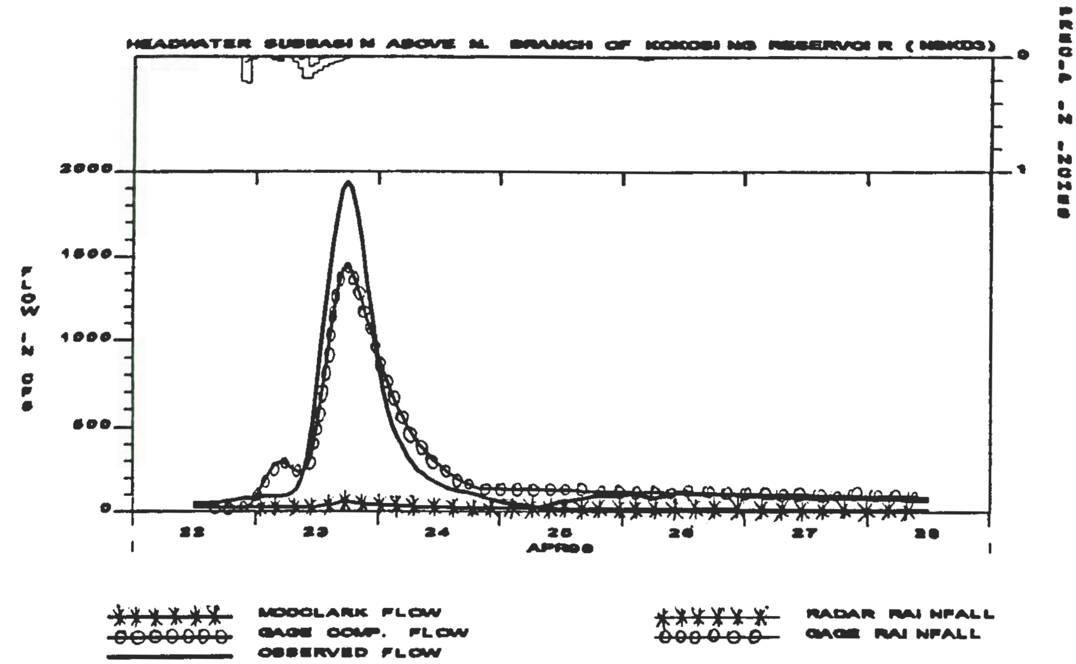

Figure 8.1. Simulations for the North Branch Kokosing reservoir subbasin in the Muskingum river basin, Ohio (reprinted from Hydrologic Engineering Center, 1996c, p. 23)

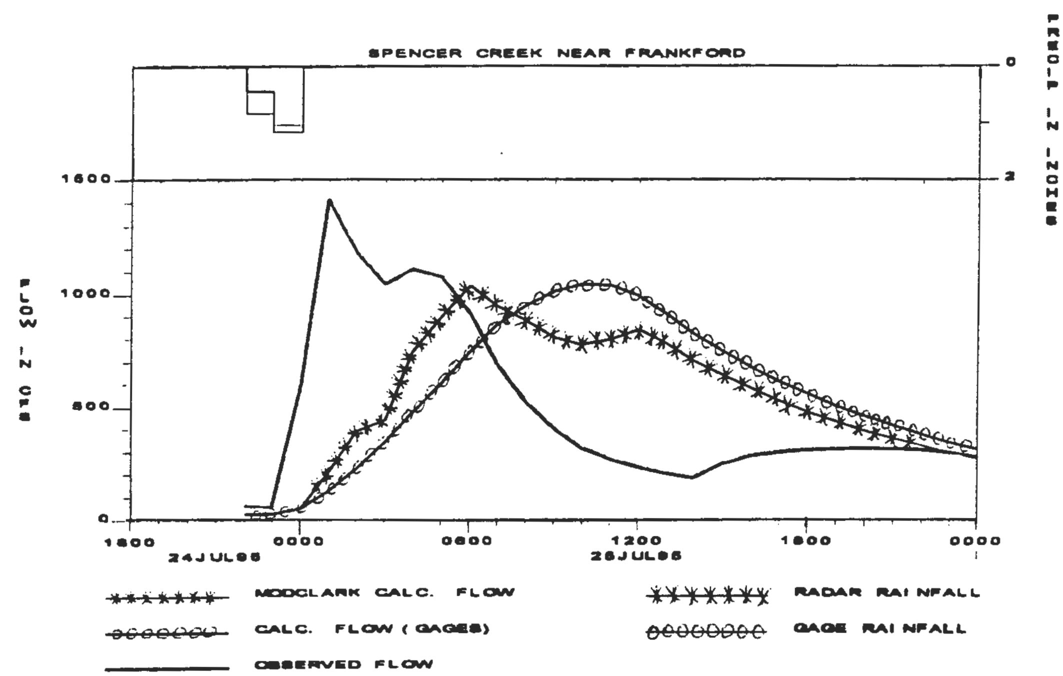

Figure 8.2. Simulations for Spencer Creek in the Salt river basin, Missouri (reprinted from Hydrologic Engineering Center, 1996a, p. 22)

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Precipitation data is now available in a format that did not exist before. One of the greatest advantages of radar-estimated rainfall is that it offers continuous areal coverage over the radar's coverage umbrella. The NEXRAD coverage is nearly total over the United States (National Weather Service, unknown date). This constitutes a vast improvement over what was available in the past. Raingage networks being sparse over some areas of the country, this new availability of precipitation data is an improvement. If there are no raingages over a particular basin or very few, the availability of NEXRAD rainfall estimates makes it possible to try and predict if a flood is going to occur. This, of course, implies the necessity for radar error sources to be minimized.

Compared to radar estimates, raingage estimates offer the advantage of being ground-based. A major problem, however, with isolated raingages is that they often miss significant precipitation. Many times, a rain shower will simply miss the bucket. It is also possible that several types of gages are used, with different levels of reliability, and different techniques for gathering and estimating rainfall. Errors can thus be easily incorporated in the collected data. Raingage estimates are most useful when the radar is subject to significant errors such as hail contamination, and when precipitation covers a large area uniformly.

It thus appears that merging both systems should enable us to obtain the point accuracy of gages and the spatial coverage of radar. A combination of both is probably the best situation for modeling. It is very likely that, in the future, when many raingage networks and radars come on-line, the data will be better corrected, resulting in reliable hourly estimates in most areas. It goes without saying that the technology has not reached this point yet, and that both data types will have to have profound quality control. For flood prediction purposes in the Squaw Creek basin, the City of Ames should ideally be able to rely not only on its raingage data, but also on radar data for detecting spotty rain showers. The current five raingages installed by the City in the basin provide quality input to the HEC-1 model. However, clouds can still spare the gages, incorporating errors in the predictions. NEXRAD's advantage is certain. Combining gage data with radar information can lead to improved precipitation estimates over the gaged area.

As mentioned previously, NEXRAD radar data can be commonly found in the form of images. The availability in digital format is a totally different

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issue. The problems encountered in this project in obtaining the radar data revealed the difficulty in getting NEXRAD rainfall data in digital format. In fact, this difficulty constitutes an obstacle.

Several programs are available -- particularly on the Internet -- for the display of NEXRAD images. But there are no programs permitting the retrieval of the digital data corresponding to these visual displays. One can, for example, obtain stage II digital data on 8 mm tapes from the NCDC. However, the NCDC does not provide any software support for the extraction of such data. The fact that NEXRAD data is a brand new rainfall data type most likely accounts for this lack of associated software. Not many people are currently modeling using this type of data, so the pressure on the NCDC may not be high. However, it will probably be needed to remedy this lack of product use and software support in a near future because the water resource community will certainly be using this type of data more and more. The current situation is not favorable for the use of NEXRAD data in hydrologic modeling.

Another issue concerning the use of gridded precipitation is the georeferencing of the precipitation cell coordinates. NEXRAD uses the HRAP system, which is well suited for the NWS, in a polar stereographic conformal projection. This coordinate system is very uncommon in hydrologic applications, which makes the conversion from this system to another a necessary step (Bradley, 1998). In the Squaw Creek research work, for example, the generation of the HRAP grid over the watershed had to include its creation in the Albers projection, used in the other parts of the work. It is mainly because of the absence of uniformity of cell size within the grid that HEC is advocating the use of a standard geographic grid instead (Evans, 1996). HEC's proposed Standard Hydrologic Grid (SHG) has cells of equal area throughout it coverage, and is based on the Albers equal-area projection, probably the most commonly used equal-area projection. In the future, the NWS may have to rethink the format the HRAP grid comes in, if NEXRAD is to be widely used within the water resources community.

NEXRAD precipitation products are not widely available outside the NWS setting. One can obtain archives in digital format -- case of the Squaw Creek project -- from the RFC, but this requires a lot of memory space. In addition, some hourly data may be missing, due to technical troubles within the

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operation of the NEXRAD data. This was the case in the project here. These facts are important to take into consideration when planning a project. The archives are also not on-line, not publicly available, and difficult to obtain without some connection with the RFC. It seems that no formal archival system has been established to distribute the data either. So one can say that, up to this date, archives have not been designed to meet the needs of the water resources community (Bradley, 1998). This situation is not ideal and some organizational improvement at the level of the NWS RFC seems necessary. If the goal is to increase the use of NEXRAD precipitation in real-time and in long-term planning, better archives and availability of all stages and products are necessary otherwise this would constitute a serious impediment to the development and use of NEXRAD in water-related applications.

At this point in time, and taking into consideration the different obstacles represented by the use of NEXRAD data, rainfall-runoff modeling using this new type of rainfall data is just beginning. NEXRAD data is still not completely "mature" from a technological standpoint. Distributed models need to be tailored to the spatial characteristic of the data. This explains why, currently, the number of hydrologic modeling research projects using NEXRAD data as input is very small. This type of modeling is in fact completely new. The quasi-distributed ModClark model is a fairly recent model, and HMS is a software still in its infancy, correcting its technical problems and adding more features to its programs. This is why so many problems were encountered during this research project, particularly with the obtainment of NEXRAD data, its processing, and the use of HMS.

8.4. Other modeling difficulties

Current obstacles in using NEXRAD data for modeling have been highlighted. Besides the number of malfunctions, typical of new software, encountered when running HMS, a few other problematic aspects concerning modeling with HMS are worth mentioning. To be able to use ModClark, one needs to be able to use GIS for the determination of the cell parameter file. This entails the use of Arc/Info on a Unix platform, since the GridParm software is only tailored to this system. These skills and resources may not always be available to any water resource project team. In addition, minimum understanding and knowledge of HEC-DSS is required, whether one will

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create raingage data DSS files or need the NEXRAD data in this format. Flow data can be obtained in different ways, which requires knowledge of different program utilities in DSS, complicating data manipulation for a researcher without this type of skill. Up to this date, the HEC program permitting the conversion of NEXRAD radar data into DSS format is also not publicly available. Hence, it is clear that we stand at the beginning of the development of these techniques and software. Much needs to be done to render modeling with NEXRAD a reasonably "easy" task if the goal is to make HMS ModClark modeling common in the water resource community.

Due to the fact that NEXRAD data is not readily available in digital format and not available at all in real-time (on-line), and that there is currently no automatic link between DSS and HMS to ease the data transfer, there is no possibility, at this time, of using NEXRAD data and ModClark in a real-time manner. The research work here illustrates this and constitutes, in fact, a first step towards this direction. HEC is still at the beginning of having its Next Generation software well functioning. More improvement is needed to achieve this level of real-time flood forecasting.

The City of Ames currently operates its HEC-1 flood prediction model in a real-time manner by inputting real-time raingage data. Since they absolutely need to have a real-time model, they cannot use ModClark yet. The absence of link between this data, the DSS system and HMS, as well as the fact NEXRAD data is not available on-line and directly linked to DSS, are major impediments to using radar data in a real-time flood forecasting mode.

In addition, the ModClark model needs to undergo some calibration in the first place, so as to make it as reliable as possible. The successful application of a hydrologic model depends on how well the model is calibrated. The calibration done on the lumped Clark model might be helpful for the calibration of the ModClark model, since certain parameters are used in both models. However, hydrologic model parameters are inherently tied to the space scale at which they are estimated (Finnerty et al., 1997). The difference in spatial resolution between the two Clark models may call for some caution in infering values for the ModClark system, based on the lumped Clark one. Another important point is that calibration requires several years of historical data. According to Finnerty et al. (1997), at least 8 years of historical data are needed for simulation and comparison to observed data. An additional 8 years

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of historical data are also recommended for model verification by these authors. Currently, about 3 years of NEXRAD data would be available from the Johnston site for calibration of the Squaw Creek ModClark model. This is an insufficient length of time for calibration and validation of model parameters. It should as well be noted that out of these years of data, the number of events might be small if the year was dry or some storms too close to one another to allow for accurate modeling. Another 20 years of data are thus needed to have a reliable ModClark model. Follow-up research concentrating on this aspect is thus a necessity.

At this point in the development of the Squaw Creek ModClark model, the use of hypothetical storms had pointed out the need for a slight modification of the time parameters related to the flood flow travel, which were causing a later peak. This finding was useful because it allowed for the adjustment of the parameters involved in the finding. The possible slight overestimation of the HRAP cell travel times also indicated that their determination can be quite difficult in a flat terrain area, even though a burn-in procedure was used to refine the GIS computations. Further research is needed on the tuning of the ModClark parameters in this project because this first-generation adjustment was solely done on fictitious data. Real data is necessary to validate the findings and refine the values. In addition, a different set of these parameters might need to be determined for non-flood situations. When the banks are not full, the velocity of water is slower, which would correspond to different values for the routing parameters. Such cases need to be studied. The first-generation adjustment of the model also did not try to tune the Muskingum x value. A value of 0.2 being generally suited to streams, no attempt was made to try and adjust these values. This could however be studied in the future, to determine if an adjustment is needed there. As of now, the Squaw Creek modClark model is functioning properly, but further calibration and verification are indispensable.

The calibration done in HMS for the lumped Clark model could be named a "first-generation optimization". One reason is that refinement of the parameter values can be achieved using more historical events and verification is of course needed with additional storm data. One also needs to point out that the HMS optimizer feature might require some software improvement. Crashing was a major problem encountered during the

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optimization runs. Optimized parameter values did not always seem to reflect the actual physical characteristics of the basin, though the objective function value was then low. Several results were discarded because values seemed very unrealistic, or causing instability within the routing, while the corresponding objective function value was low. Others were also discarded because they would not improve the peak time and magnitude at all, though they were accompanied by a low objective function value. The parameters chosen in the end as best optimized did not create instability but did not seem to be the best possible values. It was very difficult and tedious to determine the minimum and maximum soft constraints. One reason might be that event though an automatic optimization was used, it still required the skills needed for a manual calibration. Manual calibration, using a trial-and-error process of parameter adjustments, requires a good deal of exerience (Sorooschian and Gupta, 1995). The logic by which parameters should be adjusted to improve the match simulated-observed is difficult to determine. In fact, an automatic optimization algorithm may try to compensate for data errors by parameter adjustments with the results that parameter values often become physically unrealistic, and give poor simulation output when applied to a period different from the calibration period (Refsgaard and Storm, 1996). It is also difficult to know when the calibration process should be terminated. The manual procedure involves a great deal of subjective judgement; different persons may obtain very different parameter values for the same basin (Sorooschian and Gupta, 1995). It was found that the HMS optimization procedure does require the same skills as a manual optimization. A lack of years of experience may have been responsible for not getting the best possible optimized values for the lumped Clark model. The problem of the algorithm compensating data errors with value adjustments leading to physically unrealistic parameter values may have played a role. Finally, the accuracy of the input data in the first place may have prevented the obtainment of a best fit between simulated and observed hydrographs.

One could also add that the data chosen for the calibration work might not have been most appropriate. Sorooschian and Gupta (1995) emphasize that little is known about what constitutes "good" calibration data. They indicate, in particular, that: "For example, if the data selected are from a relatively dry year, certain runoff processes may not be activated, therefore the model

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response will be insensitive to some of the parameters that determine the partitioning of moisture between the various subsurface and overland flow components. However, if the data selected are from a year that is so wet that the watershed remains saturated most of the time, the model response may be insensitive to other subsurface flow controlling parameters". Referring to the discussion about surface storage in the northern part of the Squaw Creek basin, one can see the difficulties in doing a reliable calibration. Maybe two types of sets of parameters, corresponding to each above situation, would be needed. More research on this topic would certainly help. The choice of June 96 for the optimization was because of a restrained possiblity of choice and might not have been the best.

For the ModClark model to be successfully implemented in the future, further work needs to be done on its calibration, based on the preliminary estimate of the parameters and the first-generation calibration performed. This is necessary because models, by definition, are only a "rough" representation of reality, only incorporating certain parameters, judged most important among the high number of physical and process factors actually involved in the basin system. The model can never be completely exact, but its functioning can be tuned to the best possible performance, independently of the quality of the input data.

8.5. Flood mitigation alternatives

Hydrologic modeling for flood forecasting appears to be a valuable tool for early warning that can allow a certain minimization of the damage to property and potential population endangering. It is one of several flood control alternatives. These measures are divided into two groups (Yevjevich, 1994a). Structural preventative measures include cloud seeding (to suppress excess rainfall), flood water management through the construction of levees, dikes, flood walls, reservoirs or detention basins, floodplain management (forest, grass and general soil erosion control to preserve the flow capacity of the river), and channel widening or diversion. Such options have been explored by the City of Ames' officials. Dam projects on the Skunk River north of Ames and near Gilbert on the Squaw Creek have been examined on several occasions, but rejected due to high cost and opposition at the local level (see Chapter 3). Controversy also exists regarding their usefulness. The same is

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true for levees that could be constructed along the stream. As far as the floodplain management aspect is concerned, the possibility of installing riparian buffer strips along Squaw Creek would be worth considering. Riparian buffer strips are vegetated areas - usually a mix of native grasses, shrubs, and trees - adjacent to a stream (Anderson and Masters, 1992; Welsch, 1991). In agricultural watersheds, such adjacent ecosystems interact closely with the agricultural fields. However, intensive agriculture has led to the clearing of these natural corridors along the streams: this is one of the reasons modern agriculture is associated with the acceleration of the streamflow in the landscape. The riparian zone plays a critical role in the watershed's hydrology because it collects all water outputs from the drainage area (Smith, 1992; Welsch, 1991). Most research to date on riparian zones has found high infiltration rates, which indicates that a non negligible portion of the surface runoff from the upland fields gets infiltrated and lost to subsurface flow (Inamdar and Dillaha, 1994; Peterjohn and Correll, 1984). A reduction in flow velocity due to the increased surface roughness within the vegetation in the buffer has also been noted. Such attributes cannot be put aside when contemplating possibilities for flood potential reduction. Installing buffers strips along Squaw Creek might thus represent one sure way of reducing the speed with which water flows through the watershed, hence making the basin less flashy. Up to now, no project involving the installation of riparian buffers strips has been attempted. It would require the establishment of precise planning and maintenance guidelines so as to ensure successful results (Jeanne, 1994; Schultz et al., 1996). So far, the City seems to be focusing on non-structural flood control alternatives.

Non structural preventative measures include the forecasting of incoming floods from modeling and government incentives. Government incentives deal with the regulation of land use in the floodplain and would seem to carry a heavy weight in the total damage caused by a flood event. It is clear that the forecasting of flooding events does not deal with the source of the problem, primarily the intensive human use of floodplains. Human use of land increase flood risk in two ways. First, the filling of wetlands, dredging or channelizing of rivers, and urban development anywhere in a watershed increase the speed and force with which rainfall flows across the land and into rivers. Second, the intensive use of floodplains for agriculture, transportation,

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and residential development exposes more and more valuable property to damage from flooding (Montgomery, 1986). Faber (1996) relates an interesting parable that a farmer living along the Missouri river told him. The farmer compared life along the river to a bag containing 99 clear marbles and 1 black marble. Every time you pull out one of these marbles and it is black, you have a 100-year flood. After each draw, you put all 100 marbles back into the bag and shake it up. You could pull the black marble out again two or even three times in a row. Floodplain development increases the number of black marbles in the bag. Local juridictions and landowners' actions up and own the river system also take an active role in this process. Anyone who fills a wetland, improves field drainage, builds or raises a levee, paves a parking lot, or channelizes a stream is eventually pulling out a clear marble and returning a black one, gradually increasing everyone's chances of getting a black marble. In the real world, the accumulation of uncoordinated land use decisions across a watershed means that a house considered likely to flood once every 100 years when first bought may be likely to flood every 50, 30 or 10 years by the time the mortgage is paid off (Faber, 1996). Since virtually all land belongs to some watershed, all land use decisions have some potential impact on flooding. This is where the difficulty for policy-makers resides.

Planning and zoning within the floodplain should focus on eliminating some damage centers. Some land use, such as certain kinds of agriculture, railroads that must load and unload freight from different locations, may always have to be located in floodplains. But the relocation of endangered private houses and business buildings that are at risk of flooding or have already been flooded could help alleviate the flood threat and lower disaster relief costs. However, federal support towards this direction is currently lacking. It is described by the following quote: "The market is saying you're nuts to live in a floodplain, but the Federal Government is saying it's not only OK, but we'll make it affordable" (Faber, 1996). The National Flood Insurance Program (NFIP), administered by the Federal Emergency Management Agency (FEMA) (FEMA, 1997b), provides no incentives for floodplain management. By waiving the requirement that uninsured people purchase flood insurance as a prerequisite for receiving federal disaster assistance, the NFIP probably encourages further development in flood-prone areas by lowering owners' risk that floods would ruin them financially (Faber, 1996).

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Buying out people's homes and relocating them on higher ground is a costly solution. Federal grants are available, but require local governments to provide a matching amount of money. The practice is thus limited. In Ames, for instance, about 30% of the city's total land area is prone to flooding (Armour, 1994). Acquiring all concerned buildings would be unrealistic. Moreover, the City does not have enough funds to cover such high costs. Only a few homes in the flood-prone area have been bought (O'Donnell, 1994).

Possibly no one should have ever started building in the floodplain. But this is a result of things done as a civilization and one cannot change the past. What might appear to be in the realm of possibilities is a regulation of current building in the flood-prone area. There seems to be a growing use of the floodplain for commercial and residential use in Ames. The land is available and less expensive, so the demand for it is high. Requiring these property owners to elevate their buildings above the flood stage is a good idea, but does not eliminate the risk of flooding by any means. Current overall vulnerability to flooding has probably increased or is at least the same as before. This is due to the amount of development already there, and to the fact that population growth and urbanization are so quick that adequate planning and regulations cannot be established soon enough to prevent unwise use of floodplain areas. In that sense, eliminating all building in the city's floodplains would be unrealistic. Ames local officials should try and concentrate their efforts towards strictly regulating such land use. Such a strict regulation should reflect a "wise" floodplain management plan. It should aim at achieving a reduction in the loss of life, disruption and damage caused by floods, and the preservation and restoration of the natural resources and functions of the floodplains, which in turn lessen damage potential. Management ordinances could require that buidlings be elevated above a certain elevation corresponding to the 100-year flood. Floodproofing homes and businesses -by installing water-tight seals or barriers, building with water-resistant materials...- need to be encouraged by some community-sponsored programs.

One needs to remember that no matter what flood-control measures are set up, Mother Nature cannot be fooled. As long as there are going to be rains, there are always going to be some floods. And people are going to get flooded because of where they are located. But they are still going to build and that is probably as natural as flooding itself. The best protection might be to pray that

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it does not rain hard, which is probably the most cost-effective solution. What can surely be done is to raise people's awareness of all the aspects of floods. Public education about the continuing or increasing risks of buiding in the floodplain, how to react when faced with a flood, etc., would need to be incorporated in the City's flood management approach. Floodproofing the buildings located in the floodplain, and regulating land use in the flood-prone areas also are things that can be done. Planning a project that would establish vegetated buffer strips along Squaw Creek and decrease the flood potential could be another feasible option. Finally, the use of a computerized flood-warning system is an excellent flood-control tool when risk to life and property cannot be eliminated. Its full operationality is critical and investing in its refinement, particularly its calibration, is essential.

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CHAPTER 9. CONCLUSIONS AND PERSPECTIVES

Completion of this research project showed that the future of the use of HEC-HMS, ModClark, and NEXRAD data is promising because, even though the technology is currently not at its full speed, they represent a new era in the history of hydrology. Currently, no other quasi-distributed or distributed model using gridded precipitation like ModClark data are in use -- the use of raingage data is more common --, so the novelty aspect of this research is obvious. The high resolution of this precipitation data type is unprecedented and affords new oppotunities for increasing the spatial detail with which rainfall-runoff processes are simulated. There is no doubt that the use of NEXRAD is an essential complement to raingages. Similarly, the new HEC-HMS will see more and more applications within the water resources field.

The major advantage of the ModClark model is that it allows the use of NEXRAD data for any basin. This is of particular importance when considering the fact that many basins in the country have few or no raingages available to obtain precipitation data, whereas the coverage of NEXRAD is almost nation wide. A flood prediction can now be made in such cases if a ModClark model is developed for the basin in question. The development of the model on a flat terrain -- in particular the application of the GridParm procedure with a burn-in -- like central Iowa demonstrated that the ModClark model can be developed anywhere, even on flat areas. ModClark can thus ease the use of new technologies like NEXRAD, while not being drastically different from current known watershed models.

The work done here really represents a major step towards real-time flood forecasting. But there are still obstacles barring full implementation of ModClark as a real-time flood forecasting tool. The actual rainfall-runoff model has been developed but additional components and program enhancements are needed for full forecasting realization. As of now, there is no direct link between HEC-HMS and HEC-DSS to allow for easy input of rainfall data: this automation is needed. NEXRAD data is also very difficult to obtain, particularly in digital format. So the NWS would need to review its data management structure at this level. There is also no possibility of

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obtaining NEXRAD data on-line, which might be a pressing need in the near future within water resource community users.

The main findings from this particular Squaw Creek research study were the following. HMS simulations with raingage data were satisfactory in terms of peak timing. Only the June 96 flood's predicted flow was close to the actual. The overprediction for the July 96 and 97 events were most likely caused by inaccurate raingage data, raingages being unable to capture the spatial distribution of a rainstorm. ModClark simulations with radar data led to predicted flows either off in magnitude by 85%, or early or late by about 12 hours. It was however clear that the radar captured the spatial distribution of the rainfall better than the raingages. The subsequent analysis of radar data revealed that rainfall data was not always present at the right times, and not in intensities reflecting reality. The testing of the functioning of HMS with fictitious radar data indicated that HMS ModClark was functioning well and allowed the detection of a slight delay in the time of peak. These "erractic" results of the radar data simulations led to the following conclusion. Major improvements on the quality of NEXRAD data (improvement of algorithms to control the data quality; corrections with raingage data) are a necessity, for the use of this data in hydrologic modeling at this point in time leads to totally unreliable high water and flood predictions. This conclusion is similar to those drawn from both the Salt River basin and the Muskingum River basin ModClark applications by HEC. Some commonalities have surfaced from these studies and the Squaw Creek project. It is evident that Stage I radar rainfall is of limited value for flood forecasting. Runoff hydrographs generated using Stage I data can be grossly in error compared to observed hydrographs, and even compared to hydrographs generated using gaged rainfall. The trend is that Stage I radar rainfall can be very inaccurate.

The ModClark model, in its HMS environment, would also probably benefit from certain improvements. A gridded loss rate curve number feature might lead to better simulation results. The installation of a continuous soil moisture accounting system may help model each situation with more adequacy. The model also does not take into account evapotranspiration and subsurface flow, which might be useful. The addition of a reservoir type of component to the model might also help take into account certain important phenomena of surface water storage occurring in the northern portion of the watershed.

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More research is needed on this aspect. The HMS software itself can use refining for its future versions because several malfunctioning problems were discovered at all stages of this research project. In addition, the application of the GridParm procedure for other basins would benefit from the incorporation, in its set of programs, of an automation for the burn-in procedure, necessary for basins on flat terrain.

Follow-up research is needed concerning the calibration of the ModClark model. A model can only be reliable if its calibration has been the subject of extensive study, along with an adequate amount of verification runs. The first-generation optimization done on the lumped Clark model requires further work so its use as a comparison tool for runs with ModClark can be more reliable. It may also help define better values for the Clark parameters. As far as the adjustment work done on the ModClark model is concerned, it does need storm data to be tested on because all adjustments were made on fictitious radar data. It however constitutes a preliminary adjustment of the ModClark model. NEXRAD being new, little data is currently available. One thus needs to reasonably wait for another 10 years of data before the ModClark model can be reliable. Ideally, another 10 years of data are also needed to verify these calibration results. Once Stage III is reliably ground-truthed with raingage data, it will also be interesting to use it in the simulations.

The Squaw Creek ModClark model was developed so the three subbasins' outlets would correspond to the three gage stations used by the City of Ames. Once HEC-HMS has the additional feature of generating stage discharge curves incorporated in its system, the ModClark model will then be useful to the City of Ames for flood predictions. This will of course only occur when a relative confidence in the quality of radar data will have been securely established. So far, the City officials still use their HEC-1 model. Floodproofing buildings in the sensitive zones and regulating land use in the floodplain are the other best flood mitigation alternatives.

In conclusion, developing the Squaw Creek modClark model was a challenge, the necessary price to pay when one deals with cutting-edge technologies. Logistic-related obstacles were numerous. Major ones included the obtainment of radar data, its processing, its analysis, and the problems encountered with the GridParm and HMS software. Overall, HMS and the ModClark model function well, and it is the quality of NEXRAD data that is at

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fault. One needs to separate the evaluation of the method from that of the radar rainfall product used. The development of the model constitutes a real step forward, for only calibration and verification are now needed for the reliable use of it for flood forecasting, once all technological aspects will have been taken care of (automatic link to DSS, on-line NEXRAD). One however needs to wait and hope that the improvement of the quality of radar data will happen soon as well as the increased availability of all the data stages, for their use in hydrologic modeling at this point leads to unreliable results. What is interesting to note is that as improved radar rainfall products become available, they can be used in the current ModClark capability without modifications.

Finally, one essential consideration about modeling should be borne in mind. Rainfall-runoff modeling is a difficult and challenging task because of the complexity of the physical processes within the hydrologic cycle, the heterogeneous characteristics of the watershed, and the uncertainty associated with model inputs. The skills of the modeler can even be more important than the model itself. Loague and Freeze (1985) have stated the following, which is very true: " In many ways, hydrologic modeling is more an art than a science, and it is likely to remain so. Predictive hydrologic modeling is normally carried out on a given catchment using a specific model under the supervision of an individual hydrologist. The usefulness of the results depends in large measure on the talents and experience of the hydrologist and his understanding of the mathematical nuances of his particular model and the hydrologic nuances of his particular catchment." This needs to be taken into account when doing flood forecasting modeling. Modeling results can be valuable information which, when considered in relation to the current state of the basin and the meteorological forecasts, can aid in the making of reasonable flood warning decisions. However, there will always be significant uncertainty associated with model predictions, and careful interpretation of model results is essential.

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APPENDIX A. MODCLARK BASIN CHARACTERISTICS INPUT FILE

```
SUBBASIN: A
GRIDCELL: 615 519 36.61  4.57
GRIDCELL: 614 519 34.17  0.73
GRIDCELL: 614 518 29.89 15.00
GRIDCELL: 615 518 24.21 10.72
GRIDCELL: 613 518 28.06  2.77
GRIDCELL: 613 517 24.26 10.12
GRIDCELL: 616 518 24.19  1.36
GRIDCELL: 614 517 23.92 18.25
GRIDCELL: 615 517 19.29 18.25
GRIDCELL: 616 517 16.78  5.58
GRIDCELL: 613 516 21.67 14.91
GRIDCELL: 614 516 17.19 18.24
GRIDCELL: 615 516 13.55 18.24
GRIDCELL: 616 516  9.55 18.08
GRIDCELL: 617 516  8.30  5.97
GRIDCELL: 613 515 22.14 11.30
GRIDCELL: 614 515 17.07 14.93
GRIDCELL: 615 515 10.20 16.65
GRIDCELL: 616 515  4.52 17.83
GRIDCELL: 617 515  4.78  5.29
GRIDCELL: 615 514  9.62  0.02
GRIDCELL: 615 514  8.72  0.19
GRIDCELL: 616 514  6.57  0.73
GRIDCELL: 616 514  0.79  0.21
END:
SUBBASIN: B
GRIDCELL: 617 516 17.83  0.13
GRIDCELL: 617 515 15.14 12.15
GRIDCELL: 614 515 28.71  2.87
GRIDCELL: 618 515 14.60  7.45
GRIDCELL: 614 514 27.71 10.93
GRIDCELL: 615 515 26.18  1.58
GRIDCELL: 615 514 24.10 18.01
GRIDCELL: 616 515 17.51  0.36
GRIDCELL: 616 514 17.33 17.28
GRIDCELL: 616 515 14.75  0.04
GRIDCELL: 617 514 11.92 18.21
GRIDCELL: 618 514 10.44 18.21
GRIDCELL: 619 515 11.97  0.22
GRIDCELL: 619 514  8.96  8.74
GRIDCELL: 614 513 28.21  5.25
GRIDCELL: 615 513 24.04 15.71
GRIDCELL: 616 513 18.13 17.99
GRIDCELL: 617 513 10.71 18.20
GRIDCELL: 618 513  5.30 18.20
GRIDCELL: 619 513  4.54  6.54
GRIDCELL: 615 512 24.84  1.43
GRIDCELL: 616 512 24.40  0.14
GRIDCELL: 616 512 15.90  1.16
GRIDCELL: 617 512 11.28  3.83
GRIDCELL: 618 512  4.62  3.28
GRIDCELL: 619 512  0.85  0.38
END:
SUBBASIN: C
GRIDCELL: 616 513 23.84  0.22
GRIDCELL: 616 512 22.81 16.90
GRIDCELL: 615 512 29.75  3.74
GRIDCELL: 617 512 16.73 14.36
GRIDCELL: 618 512 11.13 14.91
GRIDCELL: 619 512  5.43 14.14
GRIDCELL: 615 511 29.35  0.03
GRIDCELL: 616 511 22.28  2.93
GRIDCELL: 620 512  2.32  1.32
GRIDCELL: 617 511 16.33 12.48
GRIDCELL: 618 511  9.59 12.47
GRIDCELL: 619 511  4.28  8.74
GRIDCELL: 620 511  0.65  0.56
END:
```

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APPENDIX B. OVERVIEW OF THE NEXRAD SYSTEM

The following overview of the NEXRAD system is not exhaustive. It only contains important material relevant to rainfall-runoff simulation knowledge using the ModClark method.

The Weather Surveillance Radar -- 1988 Doppler (WSR-88D) -- system is the product of the Next Generation Weather Radar (NEXRAD) program, a joint effort of the US Departments of Commerce, Defense and Transportation (Cappelletti et al., 1996; Crum and Alberty, 1993; Crum et al., 1993; Shedd and Fulton, 1993). Installed WSR-88D systems provide Doppler capabilities -in particular the measurement of the velocity of the target-, increased receiver sensitivity, and real-time display of base and derived products that enable forecasters to improve the detection of and give greater advanced warning of severe weather events. Currently, work is under way at the NWS headquarters, the National Severe Storms Laboratory (NSSL) and the WSR-88D Operational Support Facility (OSF) on a wide array of tasks to integrate the WSR-88D functionality into the existing computer platform (National Weather Service, 1997; News and Notes, 1996; Smith et al., 1996).

Unlike older radars, the WSR-88D system is made up of several computer controlled equipment groups which perform unique functions in the overall operation of the radar. The concept behind the WSR-88D system is that it estimates the precipitation rate from the sensed reflectivity. The NEXRAD system is composed of three major components: the Radar Data Acquisition (RDA), the Radar Product Generation (RPG) and the Principal User Processor (PUP) (Crum and Alberty, 1993; Crum et al., 1993; National Weather Service, unknown date; Rinehart, 1991). The RDA is the origination point of the radar data and comprises four subcomponents: the transmitter -- pulse generation and transmission --, the antenna -- broadcasting signal and intercepting returning energy --, the receiver -- amplifying the signal intercepted by the antenna --, and the signal processor -- suppressing ground clutter and converting the signal into digital data. The RDA thus acquires and processes Doppler radar data before distribution to the RPG. The RPG is a multi-function unit that processes these base data to produce weather products

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required by meteorologists. These products include base products -- reflectivity and velocity based products... -- and derived products produced from the digital data with the use of algorithms. Products are sent to users via narrow band communications. PUPs are the hardware and software used to acquire, process and store the products received from the RPG. The PUP workstation is where WSR-88D products are displayed and manipulated by meteorologists.

WSR-88D radars provide average precipitation measurements on the 4 km Hydrologic Rainfall Analysis Project (HRAP) grid defined by the NWS (Reed and Maidment, 1995). The HRAP cell coordinates are defined in the image plane of a secant polar Stereographic map projection on a spherical, earth-centered datum of radius 6371.2 km. The secant polar Stereographic projection has a standard latitude of 60° North and a standard longitude of 105° West. The fact a conformal map projection is used to create different cell sizes ranging from 3.5 to 4.5 km (Evans, 1996). Each radar has a 230 km range and the precipitation data generated can be for 1-hour, 3-hour or storm total precipitation durations (Shedd and Fulton, 1993; National Weather Service, unknown date).

The NWS has defined different stages of precipitation data processing for operational use (Crum, 1995; Crum et al., 1993; Fulton, 1997; Hudlow et al., 1991; National Weather Service, unknown date; Shedd and Fulton, 1993). A variety of complex processes is involved with each NEXRAD stage. Stage I processing, mainly the integration of rainrate intensity maps over time to produce hourly rainfall accumulations, is performed at the actual radar site. Stage I is a radar-only precipitation estimate obtained from the receiver. Stage II is processed at a NWS Warning and Forecast Office (WFO). These data are the base digital data produced by the signal processor and are based on Stage I hourly digital precipitation data. These data are then transmitted to the RPG for processing by meteorological and hydrological analysis algorithms. In the future, the NWS plans on doing further Stage II processing by using satellite and raingage data to detect and eliminate errors in NEXRAD data associated with anomalous propagation or other data contamination not detected during stage I processing. Radar and raingage data would then be merged to form an optimal "multisensor" estimate rainfall using different techniques depending on the raingages' location and the storm type. Stage III data is a product data created at the level of the RPG, and is not a base data like Stages I and II.

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Stage III processing takes place at the RFC and involves the incorporating and mosaicking of Stage II precipitation data from each radar in the RFC area onto a common grid. This precipitation product is usually constructed for major river basins and is destined to be used by the NWS for operational streamflow forecasting. Within this stage, there is possibility for more data refinement. The forecaster is given the capability to assess the quality of both the radar estimated precipitation and the precipitation gage data to make modifications to the data as deemed appropriate. Stage IV, a relatively new product not available everywhere, corresponds to a post analysis on a PUP of hourly stage III rainfall data to generate a national mosaic of regional RFC hourly rainfall on the 4 km grid. It is produced at the National Centers for Environmental Prediction (NCEP).

Because of the novelty aspect of the WSR-88D system, all the different data stages are not yet available for all of the NEXRAD sites as they are still the subject of research work. Level I radar data is probably the most commonly available stage of precipitation processing to date. The future availability of the other stages will further improve river streamflow forecasts, flash flood warnings, reservoir operations and other water management activities. In particular, stages III and IV will provide newer and useful precipitation estimates for input to many hydrologic and runoff models.

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OFFSET: 1.00

DD: 3 TYPE: 001 RATING NO: 08
START DATE/TIME: 10-01-92 (0015)

LATITIDE 420121 LONGITUDE 0933745 DRAINAGE AREA 204.00 DATUM 881.00 STATE 19 COUNTY 169
LAST UPDATED BY VEMILLER ON 02-10-1994 @ 10:50:41

GAGE HEIGHT (FEET)	DISCHARGE IN CUBIC FEET PER SECOND						(EXPANDED PRECISION)				DIFF IN Q PER TENTH FT
	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	
1.30	2.900	3.182	3.482	3.799	4.135	4.489	4.862	5.255	5.667	6.101	3.654
1.40	6.554	7.030	7.527	8.046	8.587	9.152	9.740	10.35	10.99	11.65	5.786
1.50	12.34	13.05	13.79	14.55	15.34	16.16	17.01	17.89	18.79	19.72	8.350
1.60	20.69	21.68	22.70	23.75	24.84	25.95	27.10	28.28	29.49	30.74	11.33
1.70	32.02	33.33	34.68	36.06	37.48	38.94	40.43	41.95	43.51	45.11	14.73
1.80	46.75	48.43	50.14	51.89	53.69	55.52	57.39	59.30	61.25	63.25	18.53
1.90	65.28	67.36	69.48	71.64	73.84	76.09	78.38	80.72	83.10	85.53	22.72
2.00	88.00	90.30	92.65	95.02	97.44	99.89	102.4	104.9	107.5	110.1	24.70
2.10	112.7	115.4	118.1	120.9	123.7	126.5	129.4	132.3	135.3	138.3	28.60
2.20	141.3	144.4	147.5	150.7	153.9	157.1	160.4	163.8	167.1	170.5	32.70
2.30	174.0	177.3	180.6	184.0	187.4	190.8	194.3	197.8	201.4	205.0	34.60
2.40	208.6	212.2	215.9	219.7	223.5	227.3	231.1	235.0	238.9	242.9	38.30
2.50	246.9	251.0	255.1	259.2	263.3	267.5	271.8	276.1	280.4	284.7	42.20
2.60	289.1	293.6	298.1	302.6	307.1	311.7	316.4	321.1	325.8	330.6	46.30
2.70	335.4	340.2	345.1	350.0	355.0	360.0	364.2	368.4	372.7	376.9	45.80
2.80	381.2	385.5	389.9	394.2	398.6	403.0	407.5	412.0	416.4	421.0	44.30
2.90	425.5	430.1	434.7	439.3	443.9	448.6	453.3	458.0	462.7	467.5	46.80
3.00	472.3	477.1	481.9	486.8	491.7	496.6	501.5	506.5	511.4	516.5	49.20
3.10	521.5	526.6	531.6	536.7	541.9	547.0	552.2	557.4	562.7	567.9	51.70
3.20	573.2	578.5	583.8	589.2	594.6	600.0	604.8	609.6	614.4	619.3	50.90
3.30	624.1	629.0	633.9	638.8	643.7	648.7	653.6	658.6	663.6	668.6	49.50
3.40	673.6	678.7	683.8	688.8	693.9	699.0	704.2	709.3	714.5	719.6	51.20
3.50	724.8	730.0	735.3	740.5	745.8	751.0	756.3	761.6	767.0	772.3	52.90
3.60	777.7	783.0	788.4	793.8	799.3	804.7	810.2	815.6	821.1	826.6	54.40
3.70	832.1	837.7	843.2	848.8	854.4	860.0	864.6	869.3	874.0	878.6	51.20
3.80	883.3	888.0	892.7	897.4	902.1	906.8	911.5	916.3	921.0	925.8	47.20
3.90	930.5	935.3	940.0	944.8	949.6	954.4	959.2	964.0	968.9	973.7	48.00

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	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	
4.00	978.5	983.4	988.2	993.1	997.9	1003	1008	1013	1017	1022	48.50
4.10	1027	1032	1037	1042	1047	1052	1057	1062	1067	1072	50.00
4.20	1077	1082	1087	1092	1097	1102	1107	1112	1117	1122	50.00
4.30	1127	1132	1137	1142	1147	1153	1158	1163	1168	1173	51.00
4.40	1178	1183	1189	1194	1199	1204	1209	1214	1220	1225	52.00
4.50	1230	1234	1239	1243	1247	1251	1256	1260	1264	1269	43.00
4.60	1273	1277	1282	1286	1290	1294	1299	1303	1307	1312	43.00
4.70	1316	1320	1325	1329	1333	1338	1342	1346	1351	1355	43.00
4.80	1359	1364	1368	1373	1377	1381	1386	1390	1394	1399	44.00
4.90	1403	1407	1412	1416	1421	1425	1429	1434	1438	1443	44.00
5.00	1447	1451	1456	1460	1465	1469	1473	1478	1482	1487	44.00
5.10	1491	1496	1500	1504	1509	1513	1518	1522	1527	1531	45.00
5.20	1536	1540	1544	1549	1553	1558	1562	1567	2571	1576	44.00
5.30	1580	1585	1589	1594	1598	1603	1607	1611	1616	1620	45.00
5.40	1625	1629	1634	1638	1643	1647	1652	1656	1661	1665	45.00
5.50	1670	1674	1678	1681	1685	1689	1693	1697	1700	1704	38.00
5.60	1708	1712	1715	1719	1723	1727	1731	1734	1738	1742	38.00
5.70	1746	1750	1753	1757	1761	1765	1769	1772	1776	1780	38.00
5.80	1784	1788	1791	1795	1799	1803	1806	1810	1814	1818	38.00
5.90	1822	1825	1829	1833	1837	1841	1844	1848	1852	1856	38.00
6.00	1860	1863	1867	1871	1875	1879	1882	1886	1890	1894	38.00
6.10	1898	1901	1905	1909	1913	1917	1920	1924	1928	1932	38.00
6.20	1936	1939	1943	1947	1951	1955	1958	1962	1966	1970	38.00
6.30	1974	1977	1981	1985	1989	1993	1996	2000	2004	2008	38.00
6.40	2012	2015	2019	2023	2027	2031	2034	2038	2042	2046	38.00

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	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	
6.50	2050	2053	2057	2061	2065	2069	2072	2076	2080	2084	38.00
6.60	2088	2091	2095	2099	2103	2107	2111	2114	2118	2122	38.00
6.70	2126	2130	2133	2137	2141	2145	2149	2152	2156	2160	38.00
6.80	2164	2168	2171	2175	2179	2183	2187	2190	2194	2198	38.00
6.90	2202	2206	2210	2213	2217	2221	2225	2229	2232	2236	38.00
7.00	2240	2244	2248	2252	2256	2260	2264	2268	2272	2276	40.00
7.10	2280	2284	2288	2292	2296	2300	2304	2308	2312	2316	40.00
7.20	2320	2324	2328	2332	2336	2340	2344	2348	2352	2356	40.00
7.30	2360	2364	2368	2372	2376	2380	2384	2388	2392	2396	40.00
7.40	2400	2404	2407	2411	2415	2419	2423	2427	2431	2435	39.00
7.50	2439	1443	2447	2451	2455	2459	2463	2467	2472	2476	41.00
7.60	2480	2484	2488	2492	2496	2500	2504	2508	2512	2516	40.00
7.70	2520	2524	2528	2532	2536	2540	2544	2548	2552	2556	40.00
7.80	2560	2564	2568	2572	2576	2580	2584	2588	2592	2596	40.00
7.90	2600	2604	2608	2612	2616	2620	2624	2628	2632	2636	40.00
8.00	2640	2644	2648	2653	2657	2661	2665	2669	2673	2678	42.00
8.10	2682	2686	2690	2694	2698	2703	2707	2711	2715	2719	42.00
8.20	2724	2728	2732	2736	2740	2744	2749	2753	2757	2761	41.00
8.30	2765	2770	2774	2778	2782	2786	2791	2795	2799	2803	42.00
8.40	2807	2811	2816	2820	2824	2828	2832	2837	2841	2845	42.00
8.50	2849	2853	2858	2862	2866	2870	2874	2879	2883	2887	42.00
8.60	2891	2895	2900	2904	2908	2912	2917	2921	2925	2929	42.00
8.70	2933	2938	2942	2946	2950	2954	2959	2963	2967	2971	43.00
8.80	2976	2980	2984	2988	2992	2997	3001	3005	3009	3014	42.00
8.90	3018	3022	3026	3030	3035	3039	3043	3047	3052	3056	42.00

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9.00	3060	3067	3074	3081	3088	3095	3102	3109	3116	3124	71.00
9.10	3131	3138	3145	3152	3159	3166	3173	3181	3188	3195	71.00
9.20	3202	3209	3217	3224	3231	3238	3245	3253	3260	3267	72.00
9.30	3274	3282	3289	3296	3303	3311	3318	3325	3333	3340	73.00
9.40	3347	3355	3362	3369	3377	3384	3391	3399	3406	3413	74.00
9.50	3421	3428	3436	3443	3451	3458	3465	3473	3480	3488	74.00
9.60	3495	3503	3510	3518	3525	3533	3540	3548	3555	3563	75.00
9.70	3570	3578	3585	3593	3601	3608	3616	3623	3631	3639	76.00
9.80	3646	3654	3661	3669	3677	3684	3692	3700	3707	3715	77.00
9.90	3723	3730	3738	3746	3754	3761	3769	3777	3784	3792	77.00
10.00	3800	3809	3819	3828	3838	3847	3856	3866	3875	3885	94.00
10.10	3894	3904	3913	3923	3932	3942	3951	3961	3971	3980	96.00
10.20	3990	3999	4009	4019	4028	4038	4048	4057	4067	4077	97.00
10.30	4087	4096	4106	4116	4126	4135	4145	4155	4165	4175	98.00
10.40	4185	4195	4204	4214	4224	4234	4244	4254	4264	4274	99.00
10.50	4284	4294	4304	4314	4324	4334	4344	4354	4364	4374	101.0
10.60	2385	4395	4405	4415	4425	4435	4446	4456	4466	4476	102.0
10.70	4487	4497	4507	4517	4528	4538	4548	4559	4569	4579	103.0
10.80	4590	4600	4611	4621	4631	4642	4652	4663	4673	4684	104.0
10.90	4694	4705	4715	4726	4736	4747	4758	4768	4779	4789	106.0
11.00	4800	4811	4822	4834	4845	4856	4868	4879	4890	4902	113.0
11.00	4913	4925	4936	4947	4959	4970	4982	4993	5005	5016	115.0
11.20	5028	5039	5051	5062	5074	5086	5097	5109	5121	5132	116.0
11.30	5144	5156	5167	5179	5191	5203	5214	5226	5238	5250	118.0
11.40	5262	5273	5285	5297	5309	5321	5333	5345	5357	5369	119.0

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	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	
11.50	5381	5393	5405	5417	5429	5441	5453	5465	5477	5489	121.0
11.60	5502	5514	5526	5538	5550	5563	5575	5587	5599	5612	122.0
11.70	5624	5636	5649	5661	5673	5686	5698	5710	5723	5735	124.0
11.80	5748	5760	5773	5785	5798	5810	5823	5835	5848	5860	125.0
11.90	5873	5886	5898	5911	5924	5936	5949	5962	5974	5987	127.0
12.00	6000	6011	6023	6034	6046	6057	6069	6080	6092	6103	115.0
12.10	6115	6126	6138	6150	6161	6173	6184	6196	6208	6219	116.0
12.20	6231	6243	6254	6266	6278	6289	6301	6313	6324	6336	117.0
12.30	6348	6360	6372	6383	6395	6407	6419	6431	6443	6454	118.0
12.40	6466	6478	6490	6502	6514	6526	6538	6550	6562	6574	120.0
12.50	6586	6598	6610	6622	6634	6646	6658	6670	6682	6694	120.0
12.60	6706	6718	6731	6743	6755	6767	6779	6791	6804	6816	122.0
12.70	6828	6840	6852	6865	6877	6889	6902	6914	6926	6938	123.0
12.80	6951	6963	6976	6988	7000	7013	7025	7038	7050	7062	124.0
12.90	7075	7087	7100	7112	7125	7137	7150	7162	7175	7187	125.0
13.00	7200	7215	7230	7245	7260	7275	7291	7306	7321	7336	151.0
13.10	7351	7367	7382	7397	7412	7428	7443	7458	7474	7489	154.0
13.20	7505	7520	7533	7551	7566	7582	7597	7613	7629	7644	155.0
13.30	7660	7675	7691	7707	7722	7738	7754	7770	7785	7801	157.0
13.40	7817	7833	7849	7864	7880	7896	7912	7928	7944	7960	159.0
13.50	7976	7992	8008	8024	8040	8056	8072	8088	8104	8121	161.0
13.60	8137	8153	8169	8185	8202	8218	7234	8251	8267	8283	163.0
13.70	8300	8316	8332	8349	8365	8382	8398	8415	8431	8448	164.0
13.80	8464	8481	8498	8514	8531	8548	8564	8581	8598	8614	167.0
13.90	8631	8648	8665	8682	[illegible]99	8715	8732	8749	8766	8783	169.0

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UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

EXPANDED RATING TABLE TYPE: LOG

05470500 DATE PROCESSED: 09-01-1994 @ 11:04 BY BNATIONS

SQUAW CREEK AT AMES, IA DD: 3 TYPE: 001 RATING NO: 08

OFFSET: 1.00 START DATE/TIME: 10-01-92 (0015)

LATITIDE 420121 LONGITUDE 0933745 DRAINAGE AREA 204.00 DATUM 881.00 STATE 19 COUNTY 169

LAST UPDATED BY VEMILLER ON 02-10-1994 @ 10:50:41

GAGE HEIGHT (FEET)	DISCHARGE IN CUBIC FEET PER SECOND						(EXPANDED PRECISION)				DIFF IN Q PER TENTH FT
	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	
14.00	8800	8820	8840	8861	8881	8901	8922	8942	8963	8983	204.0
14.10	9004	9024	9045	9065	9086	9107	9127	9148	9169	9190	207.0
14.20	8211	9231	9252	9273	9294	9315	9336	8357	9378	9399	210.0
14.30	9421	9442	9463	9484	9505	9527	9548	8569	9591	9612	213.0
14.40	9634	9655	9677	9698	9720	9741	9763	9785	9806	9828	216.0
14.50	9850	9872	9894	9917	9939	9961	9983	10010	10030	10050	220.0
14.60	10070	10100	10120	10140	10160	10190	10210	10230	10250	10280	230.0
14.70	10300	10320	10350	10370	10390	10410	10440	10460	10480	10510	230.0
14.80	10530	10550	10580	10600	10620	10650	10670	10690	10720	10740	230.0
14.90	10760	10790	10810	10830	10860	10880	10900	10930	10950	10980	240.0
15.00	11000	11030	11060	11090	11110	11140	11170	11200	11230	11260	290.0
15.10	11290	11320	11350	11380	11410	11440	11460	11490	11520	11550	290.0
15.20	11580	11610	11640	11670	11700	11730	11760	11790	11820	11850	300.0
15.30	11880	11910	11940	11970	12000	12040	12070	12100	12130	12160	310.0
15.40	12190	12220	12250	12280	12310	12340	12370	12410	12440	12470	310.0
15.50	12500	12520	12550	12570	12590	12620	12640	12660	12690	12710	230.0
15.60	12730	12760	12780	12810	12830	12850	12880	12900	12920	12950	240.0
15.70	12970	13000	13020	13040	13070	13090	13120	13140	13160	13190	240.0
15.80	13210	13240	13260	13280	13310	13330	13360	13380	13410	13430	240.0
15.90	13450	13480	13500	13530	13550	13580	13600	13630	13650	13680	250.0
16.00	13700	13730	13770	13800	13840	13870	13910	13940	13980	14010	350.0
16.10	14050	14080	14120	14150	14190	14220	14260	14290	14330	14360	350.0
16.20	14400	14440	14470	14510	14540	14580	14620	14650	14690	14720	360.0
16.30	14760	14800	14830	14870	14910	14940	14980	15020	15050	15090	370.0
16.40	15130	15160	15200	15240	15280	15310	15350	15390	15420	15460	370.0

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UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

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LAST UPDATED BY VEMILLER ON 02-10-1994 @ 10:50:41

GAGE HEIGHT (FEET)	DISCHARGE IN CUBIC FEET PER SECOND .00	.01	.02	.03	.04	.05	(EXPANDED PRECISION) .06	.07	.08	.09	DIFF IN Q PER TENTH FT
16.50	15500	15540	15570	15610	15650	15680	15720	15760	15790	15830	370.0
16.60	15870	15900	15940	15980	16020	16050	16090	16130	16170	16200	370.0
16.70	16240	16280	16320	16350	16390	16430	16470	16510	16540	16580	380.0
16.80	16620	16660	16700	16740	16770	16810	16850	16890	16930	16970	390.0
16.90	17010	17050	17090	17120	17160	17200	17240	17280	17320	17360	390.0
17.00	17400	17430	17470	17500	17540	17570	17610	17640	17680	17710	350.0
17.10	17750	17790	17820	17860	17890	17930	17960	18000	18030	18070	360.0
17.20	18110	18140	18180	18210	18250	18280	18320	18360	18390	17430	360.0
17.30	18470	18500	18540	18570	18610	18650	18680	18720	18760	18790	360.0
17.40	18830	18870	18900	18940	18980	19010	19050	19090	19130	19160	370.0
17.50	19200	19240	19290	19330	19380	19420	19470	19510	19560	19600	440.0
17.60	19640	19690	19730	19780	19820	19870	19920	19960	20010	20050	460.0
17.70	20100	20140	20190	20230	20280	20330	20370	20420	20460	20510	460.0
17.80	20560	20600	20650	20700	20740	20790	20840	20880	20930	20980	460.0
17.90	21020	21070	21120	21170	21210	21260	21310	21360	21400	21450	480.0
18.00	21500	21550	21600	21650	21700	21750	21800	21850	21900	21950	500.0
18.10	22000	22050	22100	22150	22200	22250	22300	22350	22400	22450	500.0
18.20	22500	22550	22600	22650	22700	22750	22810	22860	22910	22960	510.0
18.30	23010	23060	23110	23170	23220	23270	23320	23370	23430	23480	520.0
18.40	23530	23580	23640	23690	23740	23800	23850	23900	23960	24010	530.0
18.50	24060	24120	24170	24220	24280	24330	24380	24440	24490	24550	540.0
18.60	24600										

(reprinted from Tebben, 1997, pp. 61-67)

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