Development of a flood forecasting model for Flamingo Tropicana Watershed in the Las Vegas Valley

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DEVELOPMENT OF A FLOOD FORECASTING MODEL FOR FLAMINGO TROPICANA WATERSHED IN THE LAS VEGAS VALLEY

by

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Bachelor of Science
Karnataka University
1995

Master of Arts
Western Illinois University
2005

A thesis submitted in partial fulfillment of the requirements for the

Master of Science Degree in Civil Engineering
Department of Civil and Environmental Engineering
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ABSTRACT

Development of a Flood Forecasting Model for Flamingo Tropicana Watershed in the Las Vegas Valley

by

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Dr. Thomas Piechota, Examination Committee Chair
Associate Professor, Department of Civil and Environmental Engineering
University of Nevada, Las Vegas

Floods are among the most frequent natural phenomenon that occur due to excessive precipitation. Accurate and current forecasting of floods is necessary to avoid social and economic losses. Forecasting floods in an event of intense rain allows the concerned agencies to adopt appropriate measures such as warnings and evacuations and to initiate corrective and remedial efforts before disaster strikes (Chapman and Canaan, 2001).

Las Vegas has experienced rapid population growth since the 1990s. This has brought large-scale increase in impervious land surface due to the expansion of residential, commercial, and industrial area in the valley. The increase in impervious area produces more runoff volume and peak flows and consequently shortens the time that the floodwaters take to reach their peak (Hall, 1984). To effectively convey the runoff from the impervious land surface, the Clark County Regional Flood Control District.
(CCRFCD) has established regional flood control facilities. Most of the times, these facilities are adequate to protect human life and property. However, there still exist some areas of concern as recent rainfall events have caused flooding in part of the watershed thereby causing huge loss to properties and threat to lives.

This research focused on developing a hydrologic model to be used in time of intense rainfall for real-time flood forecasting. The research was carried out in the Flamingo Tropicana watershed. The existing HEC-1 flood hydrograph model of the CCRFCD was utilized to develop the flood forecasting model using the HEC-HMS software developed by United States Army Corps of Engineers. The modeling was carried out using the real-time rainfall data available through the Flood Threat Recognition System (FTRS) of CCRFCD and the gridded radar rainfall data having different resolution. The simulated hydrographs using the different rainfall data were compared with the observed data at different places in the watershed. In overall the model predicted the time to peak very well. The analysis of the results indicated that the model can be used for real-time flood forecasting in the Flamingo Tropicana Watershed. The information provided by this research can be applied to develop an integrated flood forecasting model for the entire Las Vegas Valley.
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CHAPTER 1

INTRODUCTION

1.1 Background

Floods are among the most frequent natural phenomenon that occur due to excessive precipitation. Accurate and current forecasting of floods is necessary to avoid social and economic losses. Forecasting floods in an event of intense rain allows the concerned agencies to adopt appropriate measures such as warnings and evacuations and to initiate corrective and remedial efforts before disaster strikes (Chapman and Canaan, 2001).

Flood forecasting systems include the collection of real-time rainfall data, streamflow data and the use of hydrologic and hydraulic models to predict the timing and extent of flooding. Hydrologic modeling, also called rainfall-runoff modeling, is the determination of peak flood flow and timing of the peak flow; whereas hydraulic modeling is the determination of peak water surface elevations in a channel or river. The research presented here involves the hydrologic modeling aspect of a flood forecasting system for the Las Vegas Valley.

Flooding has been a major concern in Las Vegas due to the rapid urbanization starting in the 1990s. Las Vegas has an arid climate with hot summers and relatively mild winters. Spring and Fall are the driest seasons. Winter storms are long in duration covering large areas, whereas summer storms are intense and localized. The major flood
events in Las Vegas Valley have been the result of heavy local thunderstorms (Reilly and Piechota, 2005), which are caused mainly by convective storms. Such thunderstorms create flash floods that are difficult to predict due to the absence of long-term precipitation data (Reilly and Piechota, 2005). Hydrologic modeling is often used to estimate the quantity of runoff and the time to peak. This research develops a real-time hydrologic model for a portion of the Las Vegas Valley, which will be used eventually to estimate the quantity of runoff and time to peak at any place in the watershed.

1.2 Statement of Problem

Clark County is one of the nation’s fastest growing regions with a population that has grown more than two-fold since 1990, reaching 1.8 million in 2005 (Piechota et al., 2005). This tremendous increase in population has brought large-scale developments of residential, commercial, and industrial land, which in turn has resulted in large increases in the impervious land surface area. Impervious land surfaces, such as concrete and asphalt, increase the total volume of runoff and peak flows and consequently shorten the time that floodwaters take to reach their peak (Hall, 1984). To effectively convey the runoff from the impervious land surface, the Clark County Regional Flood Control District (CCRFCD) has established regional flood control facilities consisting of lined channels and detention basins. These facilities are designed to protect human life and property. However, recent rainfall events have shown that there still exist certain areas of concern. For instance, the July 8, 1999 event produced over 1.5 inches of rainfall, within a 60-90 minute time period, in much of the Las Vegas Valley. Several gages had over 3 inches of rainfall in the same time period (CCRFCD, 2006). The damages to properties
resulting from flooding were estimated to be over $2,000,000. The August 19, 2003 event was limited to the northwest portion of the Las Vegas Valley and produced over 2 inches of rainfall in a 30-90 minute time period. This resulted in approximately $2,000,000 in damage to properties and roadways (CCRFCD, 2006). Most recently, the January 10-14, 2005 rainfall resulted in over $20,000,000 in damages to properties in the Mesquite area of Clark County (CCRFCD, 2006).

The CCRFCD, in cooperation with the United States Geological Survey (USGS) and the National Weather Service (NWS), has maintained a series of weather monitoring stations throughout Clark County as a part of its Flood Threat Recognition System (FTRS) program. The FTRS monitors current weather conditions in the Las Vegas Valley. It can identify the areas that are currently flooded but cannot be used as a forecasting tool. Therefore, there is a necessity for a reliable tool to forecast floods.

This research was the initiation of the development of a flood forecasting model for the Las Vegas Valley covering the Las Vegas Wash near the Clark County Wetland Park, which receives runoff from the entire valley. To develop an integrated model for the entire Las Vegas Valley watershed is a huge task. Therefore, the research presented here focuses on the Flamingo-Tropicana Watershed, which represents a complex watershed in the Las Vegas Valley with mixed land use, and drains into the Las Vegas Wash.
1.3 Research Questions

The overall objective of this research was to develop and test a real-time flood forecasting model for the Flamingo-Tropicana watershed in the Las Vegas Valley. To accomplish the overall objective, this research focused on the following specific research questions:

1. Can the CCRFCD Master Plan model, which is developed for hypothetical storms, be used in real-time flood forecasting?
2. Can the real-time gage precipitation data be used for accurate flood forecasting?
3. Does 1-km resolution radar rainfall data significantly improve the peak flow forecast as compared to 2-km resolution radar rainfall data?

1.4 Presentation of This Research

This thesis is divided into five chapters. Chapter 1 provides an overall introduction of the research. Chapter 2 reviews the related literature and other technical studies regarding hydrological processes and modeling. Chapter 3 describes the methodology and detailed procedures carried out to develop the hydrologic model for the Flamingo-Tropicana watershed. The problems encountered during the modeling are also discussed in this chapter. Chapter 4 summarizes the results of research obtained from modeling. Lastly, the conclusions and recommendations of this research are presented in Chapter 5.
CHAPTER 2

LITERATURE REVIEW

This chapter begins with the review of the basic watershed level hydrologic processes related to hydrological modeling. This will be followed by the review of runoff generation, runoff modeling, and runoff hydrograph. Finally, a review of recent development and application in hydrologic modeling systems will be presented.

2.1 Hydrologic Processes

Hydrologic modeling is the mathematical representation of the hydrological processes taking place at the earth surface. Hence, a clear understanding of the hydrologic processes at the watershed scale is necessary for undertaking hydrologic modeling. It is a vast area of study and a number of studies have been carried out to investigate the hydrological processes. Chow et al. (1988) provides a discussion of the hydrological processes, which is shown schematically in Figure 2-1 with a brief description of each process involved as follows:
Precipitation is the most important contributing factor for the generation of runoff in a watershed. Precipitation can take place in the form of rain, snow, hail, sleet, and dew. The amount of precipitation varies in nature both spatially and temporally. The research presented in this thesis considered precipitation only in the form of rain since it is the major factor responsible for hydrologic processes in semi-arid regions.

Rainfall is partitioned in the watershed in different forms as shown in Figure 2-1. Vegetation intercepts a fraction of the rainfall and some of this water is evaporated back
to the atmosphere. Water is also sent back to the atmosphere due to evaporation from the soil. Throughfall is the rainfall that drops on the land surface with or without being intercepted by the vegetation. Part of the throughfall is stored at the land surface as depression storage. Therefore, interception, infiltration, depression storage and evaporation together are referred to as a loss component to the watershed runoff.

The infiltrated water generally percolates deeper into the soil in a downward direction through the unsaturated subsurface layer and recharges the groundwater system. In some cases, the groundwater flows laterally into the stream as base flow. Subsurface water, in the form of interflow, also flows back to the land surface as return flow and adds to overland flow.

2.2 Runoff Generation

The purpose of this research is to forecast the peak runoff due to a rainfall event. Therefore, it is important to understand the process of runoff generation. A number of authors, for example Taborton (2003) and Dunne (1982), have discussed this topic.

A watershed is made up of rivers and the areas draining to these rivers. The amount of runoff from the watershed is mainly influenced by its characteristics, especially the physical characteristics that include landuse, soil type, antecedent soil moisture, vegetation, slope, and topography of the watershed. Overland flow and interflow (shallow groundwater) that transport water to the stream define how much runoff occurs. Thus, runoff in a watershed is from both surface and subsurface sources.

Surface runoff occurs when the precipitation rate exceeds the infiltration rate, or when the soil is saturated. The runoff due to rainfall exceeding the infiltration capacity of
soil is known as the Horton's runoff. The runoff resulting from the saturation of soil due to the groundwater rise is known as the Dunne's runoff (Dunne, 1982). Surface runoff includes overland flow, streamflow, and channel flow that occurs over the land surface due to the difference in gradient. The overland flow initially occurs as sheet flow. As it flows downward the rill flow is developed. A number of rill flows develop the streamflow, which then converges into channel flow.

The amount of rainfall that infiltrates and flows slowly on its way to the stream is known as subsurface runoff (Horner et. al., 1994). Subsurface flow includes various flows from unsaturated, perched, and groundwater flow. Unsaturated subsurface water flows vertically, while the perched subsurface water flows in a lateral direction. Perched subsurface water flow takes place where the shallow soil layer has a higher hydraulic conductivity as compared to the soil layer beneath. Groundwater flow is produced in the saturated zone, where the water is fed from unsaturated zone. Groundwater also flows downhill depending on the slope of the water table and contributes to the channel system.

2.3 Rainfall-Runoff Modeling

The development of rainfall-runoff models is attributed to the sparse number of gages in most watersheds as well as to the need to know the flow rates at ungaged locations. Rainfall-runoff models generally predict the peak flow and time to peak that are required for various hydrological design problems.

As described above, the hydrological process is complex and it is not simple to quantify the various processes involved therein. Rainfall-runoff modeling is a tool, which can be used to find an abstraction of the various processes involved in hydrological processes. Therefore a reliable rainfall–runoff model must be based on the knowledge of
various hydrological phenomena like overland flow, subsurface runoff, infiltration, evaporation etc.

The rainfall-runoff model has been in use since the 1960's to simulate the transformation of rainfall to runoff (Todini, 1988). Early models were based on empirical equations and rational methods. Both methods were applicable to predict peak flow rate in small watersheds. Later there arose the need of procedures to predict the runoff in large watersheds and hence new models were developed (Todini, 1988).

The rainfall-runoff process is an active and deep area of study with continually emerging new understanding (Tarboton, 2003). Rapid advancement of computer technology has contributed a lot in the development of rainfall-runoff models. Hydrologists have carried out considerable research using the more advanced computing techniques and complex models. As a result, a large number of models have been developed to better understand the processes. Todini (1988) provides a summary of rainfall-runoff models. The hydrologic modeling inventory compiled by the United States Bureau of Reclamation (USBR) also provides a list with a large number of state-of-the-art watershed models developed by government (federal, state, and local) agencies, universities, and private companies in the United States and elsewhere. One can refer to http://www.usbr.gov/pmts/rivers/hmi/invlist99.html for a complete list of inventory and concise summaries with information on each model. This inventory is among the first of its kind and is useful not only for the modelers but also for water resources planners and managers.

Hydrologists have categorized rainfall-runoff models depending on their specific approaches as well as their characteristics. According to Singh et al. (2002), when we
consider rainfall-runoff models, we can classify them into three major categories; physics-based, conceptual based, and empirical based. Examples of the three models are shown in Table 2-1. The physics-based rainfall-runoff models (white box) are based on laws of physics that use the law of conservation of mass, momentum, and energy to describe the hydrological processes. The equation of conservation and mass are most popularly used in current models. The physics based models can be set up with minimal historical data and they still generate reasonably accurate output (Vieux et al., 2002). The conceptual rainfall-runoff models (gray box) consider physical laws in a more simplified form than the physics-based model and use the empirical expressions to explain hydrological processes. The empirical based rainfall-runoff models (black box), do not aid in physical understanding but contain parameters that allow modeling based on simple empirical expressions.

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<td>Representative Elementary Watershed (Reggiani and Rientjes, 2005)</td>
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<td>r.water.fea (Vieux, 2001)</td>
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<td>Distributed Hydrology Soil Vegetation Model (Wigmosta, et al., 1994)</td>
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<td>2</td>
<td>Conceptual</td>
<td>Tank Model (Sugawara, 1995)</td>
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<td>Sacramento Soil Moisture Accounting (Burnash, 1995)</td>
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<td>HEC-HMS (HEC, 2000)</td>
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<td>3</td>
<td>Empirical</td>
<td>Unit Hydrograph and Rational Methods (Singh, 1988)</td>
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In terms of the spatial domain, the rainfall-runoff models can be classified as lumped, distributed, or semi-distributed models. Lumped models do not take into consideration
the spatial heterogeneity of the watershed. Instead, they are averaged or lumped by a single value. The distributed models consider the watershed as divided entities (subbasins) and account for routing. The spatial structure of the watershed is taken into account along with the spatial variability of the hydrological processes to predict the watershed response to runoff (Abbott et al., 1986). Distributed models generally produce more accurate results compared to lumped models. However, they require a large amount of data and advanced computing powers (Larson et al., 1982). A semi-distributed model is in between the distributed and lumped model. It does consider the watershed as a divided entity, but in a coarser unit as compared to the distributed model.

2.4 Runoff Hydrograph

A runoff hydrograph is a graph of water flow versus time. The rainfall-runoff model, mostly used today, makes use of the unit hydrograph to generate its output. A unit hydrograph is useful to translate the amount of runoff generated in the basin as a result of an inch of rainfall excess from the watershed.

Sherman (1932) introduced the concept of unit hydrograph based on the principle of superposition. It was one of the first tools available to hydrologists to predict entire hydrographs instead of just peak discharges (Todini, 1988). He defined the unit hydrograph as the watershed response to a unit depth of excess rainfall, uniformly distributed over the entire watershed.

Sherman’s unit hydrograph is applicable only to gaged watersheds. Unfortunately, most of watersheds are not gaged. Snyder (1938) developed the first standard unit
hydrograph called the synthetic unit hydrograph that can be used to develop a unit hydrograph even for ungaged basins (Bedient and Huber, 2001). Snyder proposed relations between the characteristics of unit hydrograph such as peak flow, lag time, and width at 50% and 75% of the peak flow (Chow et al., 1988). Furthermore, Clark (1945) provided a significant contribution to the synthetic unit hydrograph theory by proposing the unit hydrograph as the result of combination of a pure translation routing process followed by a pure storage routing process. The Soil Conservation Service (SCS), now called NRCS, the Natural Resources Conservation Service, in 1957 developed the SCS unit hydrograph method based on a dimensionless hydrograph. The SCS method works on the assumption that the unit hydrograph can be approximated by a triangle and uses a curve number to calculate the runoff from the basins.

Practical development of the unit hydrograph method was advanced by the United States Army Corps of Engineers, Hydrologic Engineering Center through the development of HEC-1 and HEC-HMS (Vieux et al., 2002). It assumed average parameters and input values for subbasins and derivation of unit hydrographs comes from gage records or from synthetic estimation techniques. It also assumes that the rainfall is uniform over the entire basin and that the basin always responds to the same degree given a unit of rainfall excess.

2.5 Geographic Information System (GIS) and Hydrologic Modeling

Hydrologic models require data, which describe the connectivity and properties of individual subbasins that govern the movement of water in the whole watershed. Since hydrologic models are based on fundamental hydrologic principles, equations, and
numerical models, they require data that is of quality and precision to assure reliable results. Increases in computing powers and uses of more advanced and efficient tools in recent years in data collection, distribution and processing have contributed much to the development of hydrologic models.

GIS has been used widely in many hydrological applications. Rapid development of GIS has played a key role in hydrological modeling to such an extent that it is difficult to envision a rainfall-runoff model without using GIS. Hydrological models have been transformed from lumped systems to spatially distributed systems because of the increased power of computers and the advent of GIS. GIS is now the most widely used available tool to derive model parameters and to develop models because of its capability to support database information and ease of analysis (Clark, 1998). Vieux (2001) has provided a methodology to develop the hydrologic modeling using GIS.

GIS has the capability to combine spatial data such as location and topography of a feature with its attribute data such as soil, landuse, and land cover for hydrologic modeling. GIS can process these large amounts of geo-spatial data along with other distributed parameters (Vieux, 2001). Such a model can accurately depict the reality that the hydrologist is trying to model. However, the GIS data must be error free and translate the data correctly into the model in order for it to simulate the hydrologic process correctly (De Roo, 1998).

GIS cannot be used alone for hydrological modeling. It has to be integrated. Charnock et al. (1996) described two levels of GIS and hydrologic model integration. The first one combines GIS and model through tight integration with each component communicating directly with each other. The second one relates GIS and model through a
programming media, in which each one is executed separately but they share the data through some links. The later form of integration of GIS and the hydrologic models is the most common (Kopp, 1996). In such models, GIS can usually be used as a pre- and post-processor tool to share the information to develop the hydrologic model (Stork et al., 1998).

2.6 Use of Radar Rainfall Data for Hydrologic Modeling

Rain gages record point rainfall data and provide fairly accurate records of rainfall in small areas where a network of rain gages usually exists. However, in large areas, the sparse distribution of the network of rain gages cannot capture the rainfall that occurs between the gages (Liu et al., 2005). It is well known that rains varies greatly in time and space. Therefore, it is not always practical for hydrologists to define the rainfall using the gage data in large watersheds. In such circumstances, radar data provides better spatial pattern of rainfall because it has greater spatial coverage and higher resolution.

Due to the technological advancement in the recent years, there have been significant developments in the use of radar rainfall data. As a result, a number of efficient tools have been developed that are mainly aimed at increasing the quality of data. A recently deployed Doppler radar system, commonly known as NEXRAD (Next Generation Weather Radar) provides weather information for much of the United States. It is also an important source of rainfall information to hydrologists for real-time flood forecasting. The radar precipitation data derived from NEXRAD has been used successfully in hydrologic modeling (Robayo et al., 2004). Kouwen (1988) also used radar precipitation...
data to develop a real-time hydrologic model and concluded that the radar data is effective in watersheds having an area up to 6,500 square miles.

Studies have shown that radar data provides more accurate rainfall estimates (Pessoa et al. (1993), but they require proper calibration. The radar rainfall data cannot be used by itself because the radar rainfall estimates are not always consistent with rainfall estimates made by rain gages (Liu et al., 2005). Better results are obtained if radar data is used with gage rainfall data. Therefore, the radar data is generally used as a supplement to, rather than a replacement for, gage rainfall data. Gage adjusted radar rainfall estimates combine the advantages of both radar and gages. The radar captures the temporal and spatial characteristics of rainfall and the gages measure the actual rain falling on the ground (Liu et al., 2005). Such gage-adjusted radar rainfall data have been used in a number of studies. James et al. (1993) developed a flood forecasting models using radar and gage rainfall data and found that the gage-adjusted radar rainfall data provided more accurate hydrographs. Sun et al. (2000) also studied the hydrographs that were developed using radar and observed rainfall data and concluded that the use of radar data improves forecasting when used in conjunction with observed rainfall data.
CHAPTER 3

METHODOLOGY AND PROCEDURES

This chapter presents the methodology and detailed procedures beginning with a brief overview of the hydrologic modeling. This is followed by the introduction of the existing Hydrologic Engineering Center-1 (HEC-1) Master Plan Update (MPU) model used in this research. Then, the step-by-step procedures involved in developing the hydrologic model are provided. The technique used for the parameter optimization is also presented. Finally, hydrologic modeling using the radar data is presented.

3.1 Overview of the Hydrologic Modeling Process

This research was carried out using the Hydrologic Engineering Center- Hydrologic Modeling System (HEC-HMS) version 2.2.2 released by the United States Army Corps of Engineers (USACE), Hydrologic Engineering Center in May 2003. HEC-HMS is new generation Windows-based software that will supersede HEC-1. It is designed to simulate the surface runoff response of a watershed to precipitation by computing the streamflow hydrographs at desired locations in the watershed; and is applicable in a wide range of geographic areas for solving the widest possible range of problems (USACE, 2000). As
shown in Figure 3-1, the HEC-HMS model requires a basin model, precipitation model, and control specifications. Additional information of the implementation of HEC-HMS in this research is presented in section 3.3.

Figure 3-1. HEC-HMS' main project definition window showing the component models (basin, meteorologic, and control specifications) required for a complete hydrologic model.

Figure 3-2 outlines the steps involved in developing the hydrologic model in this research. The basin model was created from the existing HEC-1 MPU model. The meteorologic model and control specifications were created as described in sections 3.3.2 and 3.3.3 respectively. The HEC-1 model represented the ultimate build-out condition in the watershed. However, the watershed was not fully developed during the rain events considered in this research. This difference in landuse in space and in the model causes disparity between actual and modeled flow rates. The sensitivity analysis provided new
parameter values, which were used in the model to best fit the simulated flow with the observed flow.

Figure 3-2. Steps involved in this research to develop the hydrological model.

3.1.1 Study Area

This research was carried out in the Flamingo-Tropicana Watershed located in the southwest and central part of Las Vegas Valley in Clark County, Nevada (Figure 3-3). The watershed extends from the Spring Mountain Range on the western rim of the Las Vegas Valley to the confluence of the Flamingo Wash and the Lower Las Vegas Wash. The altitude of the study area varies from 8000 feet to 1500 feet.
Figure 3-3. The nine watersheds in Las Vegas Valley addressed by the HEC-1 MPU Model: NORTH (North Basin), GOWAN (Gowan Basin), CENTRAL (Central Basin), RANGE (Range Wash), FLAM TROP (Flamingo Tropicana Washes), DUCK (Duck Creek Wash), PITTMAN (Pittman Wash), C1 (C1 Channel), LOWER (Lower Las Vegas Wash).
The total area of the Flamingo-Tropicana Watershed is approximately 215 square miles. As seen in the Figure 3-4, the 34 square miles of area in the western portion of the watershed is undeveloped mountainous region that is within the Red Rock National Conservation Area. The southwest and western region in the watershed is experiencing development due to rapid population growth. The watershed is comprised of diverse subbasins that are naturally drained, regulated or have a complex urban drainage system. Runoff in the watershed travels through a series of detention basins connected by conveyance facilities before draining to the Las Vegas Wash.

Figure 3-5. Major washes, detentions basin, storm drain channels, and rain gage stations in the Flamingo-Tropicana Watershed.
The mean annual rainfall in the study area is about 4 inches (CCRFCD, 2002). Runoff in the watershed generally flows from west to east. The Red Rock, Flamingo, Tropicana, and Blue Diamond washes originate in the western mountainous region. These major washes and several other small washes collect the runoff from the undeveloped highland. Streets and storm drain systems that receive runoff from the developed areas in the watershed also drain to the major washes. The runoffs from these washes are intercepted by regional flood control facilities. These facilities discharge the runoff into two main washes, the Flamingo Wash and the Tropicana Wash. As the washes proceed to the east, the Tropicana Wash joins Flamingo Wash and the Flamingo Wash continues to the east till it exits the watershed and proceeds to the Las Vegas Wash and Lake Mead.

3.1.2 Data Sources

This research used two types of data: the real-time rainfall data and the stream flow data. The rainfall data was used to develop the hydrologic model and the stream flow data was used to calibrate/verify the model.

The rainfall data were obtained from the CCRFCD Flood Threat Recognition System (www.ccrfcd.org/sensordata.htm). CCRFCD has established a series of weather stations in the Clark County as a part of its Flood Threat Recognition System program. The FTRS provides real-time rainfall data from 155 weather stations in Clark County. The majority of these stations are located in urbanized areas, of which 32 are located in the Flamingo-Tropicana Watershed as shown in Figure 3-7. The names of the weather stations are provided in Appendix 1. These stations automatically record and transmit the real-time rainfall data, which have been archived on a monthly basis since 1989.
Two types of flow data were used to test the model: observed flow data (time-series) and the water level data. The observed flow data were supplied by the USGS. These data were recorded for the Flamingo-Tropicana Wash at USGS gage # 094196781 located at the outlet of the Flamingo-Tropicana watershed near Nellis Boulevard. The observed flow data were based on a 15-minute time interval and were compared with the simulated flows as described in section 4.2.2.

The water level data were downloaded from the FTRS. In addition to the real-time rainfall data, FTRS also provides water level information in the channels under the same drop down window. Water level information are in fact stage data that are helpful to know about the peak water surface level in the stream. Using the stage data, the peak water surface elevation was identified. The peak water levels were then converted to peak
discharges using rating curves (see Appendix 2). The rating curve for Upper Flamingo Detention Basin and Tropicana Detention Basin were obtained from the CCRFCD. The peak flow data obtained from the stage data was used to test the performance of the model at various locations in the watershed as described in section 4.2.3.

GIS data were also used to make the necessary maps and were obtained from the website of Clark County GIS Management Office (GISMO). Finally, the research used radar rainfall data to test the hydrologic model. The radar rainfall data were obtained from OneRain, Inc. The Table 3-1 summarizes the different data used in the research.

Table 3-1 Different types of data used in the research

<table>
<thead>
<tr>
<th>Data</th>
<th>Source</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Real-time gage rainfall data (provided by FTRS of CCRFCD)</td>
<td><a href="http://www.ccrfcd.org/sensordata.htm">www.ccrfcd.org/sensordata.htm</a></td>
<td>Used to develop the hydrologic model</td>
</tr>
<tr>
<td>Flow data</td>
<td>USGS</td>
<td>Used to test/calibrate the hydrological model</td>
</tr>
<tr>
<td>Water level data (provided by FTRS of CCRFCD) and the stage discharge curve</td>
<td>Flow data downloaded from <a href="http://www.ccrfcd.org/sensordata.htm">www.ccrfcd.org/sensordata.htm</a> and stage discharge curve obtained from CCRFCD</td>
<td>Used to test/verify the model flow at different places in the watershed Used to make the necessary maps</td>
</tr>
<tr>
<td>GIS Data</td>
<td>Downloaded from GISMO of CCRFCD</td>
<td>Used to make the necessary maps</td>
</tr>
<tr>
<td>Radar Data</td>
<td>Obtained from OneRain Inc.</td>
<td>To test the hydrologic model with radar rainfall data</td>
</tr>
</tbody>
</table>
3.2 HEC-1 Master Plan Update Model

CCRFCD has published the HEC-1 Flood Hydrograph Package Master Plan model for the entire Las Vegas Valley. The model is a planning tool for the design and construction of flood control facilities. The model was originally prepared in 1986 and revised in 1991 and 1997 to account for the ongoing development in the watershed in order to insure the most accurate planning of the flood control systems. Most recently in 2002, GC Wallace, PBS&J, and Louis Berger Group updated the Flood Control Master Plan models for CCRFCD. These updated models are known as the HEC-1 MPU model.

The HEC-1 MPU model was prepared for the Las Vegas Valley Watershed that drains into the Las Vegas Wash. To facilitate the implementation of the Flood Control Master Plan, the entire Las Vegas Valley Watershed has been divided into nine individual watersheds (Figure 3-3). Each watershed was analyzed in the MPU using consistent criteria and methodology. The HEC-1 models were developed considering that the watersheds have reached ultimate build-out condition. In other words, the model assumes that all available land within the Las Vegas Valley has been fully developed. This condition is assumed considering that flood control facilities, once completed, would be able to serve efficiently in the future when the watershed is fully developed.

In the original HEC-1 MPU model, the ultimate condition is used in conjunction with the 100-year frequency flood event to establish peak flow rates and flow volumes. These peak flow rates and flow volumes are then used for the design of flood control facilities. As the model uses the 100-year design storm, it could not be used for flood forecasting which requires real-time precipitation. Therefore, this research used only the basin model
of the existing HEC-1 MPU model. New meteorologic models were created to use the real-time precipitation data available through the FTRS of CCRFCD and radar data.

3.2.1 HEC-1 Basin Model

The basin model represents the physical characteristics of the watershed. In the basin model, the watershed is represented by any combination of hydrologic elements such as subbasin, reach, reservoir, junction, diversion, sources, and sink. The development of a basin model requires the specifications of these elements and the data that controls the flow of water through these elements in the watershed.

The basin model requires setting up the parameters for calculating the basin loss, runoff transform, runoff routing, and base flow. The basin loss parameter computes the amount of rainfall lost in the subbasins due to the infiltration characteristics of the soil. The runoff transformation parameter simulates excess rainfall to runoff. The basin runoff can be computed in either a lumped or distributed basis. In the former case, precipitation and losses are spatially averaged over the basin. Whereas, in the latter case, rainfall is specified on a grid basis, and losses are calculated separately for each grid on the basin. The routing parameter is required to convey the runoff from reaches within different basins to the end of the basin. Flood routing simulates the flood movement through river reaches and reservoirs. The routing model computes the downstream hydrograph based on the upstream hydrograph by solving the continuity and momentum equation. The base flow parameter computes the amount of water lost as base flow before the runoff generates. The HEC-1 model assumes no base flow condition as it produces a more conservative peak flows for the design of flood control structures.
The basin model has various options for calculating the above parameters. As the basin model came from HEC-1 MPU model, the above parameters were already set in the model. The methodology used to set these parameters in the HEC-1 MPU model is described below.

3.2.1.1 Loss Parameter

The HEC-1 basin model uses initial/constant and SCS-CN (Curve Number) methods to calculate the precipitation loss in the basins. The initial/constant rate method needs the parameters for constant rate, initial loss, and % of impervious land. This method is used for the undeveloped basins in the Red Rock Conservation Area in the west. Since this is a conservation area, it was predicted to remain as an undeveloped area. The 1998 study carried out by the U.S. Army Corps of Engineers suggested using an initial/constant method with a constant loss rate of 0.5 inches per hour (USACE, 1988).

All the remaining areas in the watershed were assumed to undergo development and a full build-out condition is envisioned in the model. These areas use the SCS-CN method for computing the runoff from the basins. The SCS-CN method divides the rainfall into infiltration and runoff, using the empirical relationship between precipitation, soil type, land use, and antecedent moisture condition, to calculate the precipitation excess (SCS, 1986) as shown below.

\[
R = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{1}
\]

\[
I_a = 0.2S \tag{2}
\]

Combining equation I and II,

\[
R = \frac{(P - 0.2S)^2}{P + 0.8} \tag{3}
\]

Where:
R = Direct Runoff (inches)
P = Rainfall depth (inches)
S = Potential maximum retention (inches)
l_a = Initial abstraction (inches)

In practice, the value of S is determined by the following relation

\[ S = \left( \frac{1000}{CN} \right) - 10 \]  

(4)

Source of Curve Number: In equation 4, the CN is an index. Empirical analysis suggested that the CN is a function of soil group, the cover complex, and the antecedent moisture conditions. The SCS has classified more than 4000 soils into four hydrologic soil groups according to their minimum infiltration rate obtained for bare soil after prolonged wetting. The four hydrologic groups are denoted by the letters A, B, C and D each representing distinct soil group as follows:

Group A: soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist of deep, well to excessively drained sands and gravels and have a high rate of water transmission (greater than 0.30 in/hr).

Group B: soils have moderate infiltration rates when thoroughly wetted and consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15 - 0.30 in/hr).

Group C: soils have low infiltration rates when thoroughly wetted and consist of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05 - 0.15 in/hr).
Group D: soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist of clay soils with high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0.0 – 0.05 in/hr).

The values of curve number for some selected land uses are given in Table 3-2. Curve number values range from 0 to 100. If the value is 0 then no runoff is generated while if the curve number is 100 all the rainfall is transformed into runoff without any abstractions. Example: for impervious and water surfaces CN = 100; for natural surfaces CN < 100.
Table 3-2  Curve Numbers for Selected Land Uses

(U. S. Soil Conservation Service, 1986)

<table>
<thead>
<tr>
<th>Hydrologic Soil Group</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Land use description</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Cultivated land</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without conservation treatment</td>
<td>72</td>
<td>81</td>
<td>88</td>
<td>91</td>
</tr>
<tr>
<td>With conservation treatment</td>
<td>62</td>
<td>71</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td><strong>Pasture or range land</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor condition</td>
<td>68</td>
<td>79</td>
<td>86</td>
<td>89</td>
</tr>
<tr>
<td>Good condition</td>
<td>39</td>
<td>61</td>
<td>74</td>
<td>80</td>
</tr>
<tr>
<td><strong>Meadow</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Good condition</td>
<td>30</td>
<td>58</td>
<td>71</td>
<td>78</td>
</tr>
<tr>
<td><strong>Wood or forest land</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thin stand, poor cover, no mulch</td>
<td>45</td>
<td>66</td>
<td>77</td>
<td>83</td>
</tr>
<tr>
<td>Good cover</td>
<td>25</td>
<td>55</td>
<td>70</td>
<td>77</td>
</tr>
<tr>
<td><strong>Open spaces, lawns, parks, etc.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Good condition (grass cover on 75% or more of the area)</td>
<td>39</td>
<td>61</td>
<td>74</td>
<td>80</td>
</tr>
<tr>
<td>Fair condition (grass cover on 50 to 75% of the area)</td>
<td>49</td>
<td>69</td>
<td>79</td>
<td>84</td>
</tr>
<tr>
<td><strong>Commercial and business areas (85% impervious)</strong></td>
<td>89</td>
<td>92</td>
<td>94</td>
<td>95</td>
</tr>
<tr>
<td><strong>Industrial districts (72% impervious)</strong></td>
<td>81</td>
<td>88</td>
<td>91</td>
<td>93</td>
</tr>
</tbody>
</table>

| Residential |    |    |    |    |
| **Average lot size** |    |    |    |    |
| 1/8 acre or less | 65 | 77 | 85 | 90 |
| ¼ acre | 38 | 61 | 75 | 83 |
| 1/3 acre | 30 | 57 | 72 | 81 |
| ½ acre | 25 | 54 | 70 | 80 |
| 1 acre | 20 | 51 | 68 | 79 |
| **Paved parking lots, roofs, driveways, etc.** | 98 | 98 | 98 | 98 |
| **Streets and roads** |    |    |    |    |
| Paved with curbs and storm sewers | 98 | 98 | 98 | 98 |
| Gravel | 76 | 85 | 89 | 91 |
| Dirt | 72 | 82 | 87 | 89 |
3.2.1.2 Runoff Transformation Parameter

The basin model uses the SCS Unit Hydrograph method for transforming the excess precipitation to surface runoff. This method is based upon averages of unit hydrographs derived from gaged rainfall and runoff for large number of small basins. The SCS method requires a lag parameter, which is calculated using two different methods. Large basins use the U.S. Bureau of Reclamation method, and small basins use the Time of Concentration method (CCRFCD, 2002).

U.S. Bureau of Reclamation Method: This method is used to compute the lag time for basins with areas greater than one square mile using the following equation:

\[
T_{\text{lag}} = 20 K_n \left( \frac{L L_c}{S^{1/2}} \right)^{1/3}
\]  

(5)

Where:

- \( T_{\text{lag}} \) = Lag time (hours)
- \( L \) = Watershed length (miles)
- \( L_c \) = Length along longest watercourse (miles)
- \( S \) = Average slope of the longest watercourse (ft./mile)
- \( K_n \) = Manning’s roughness coefficient taken as 0.050 for all basins

Time of Concentration Method: The basins having areas less than one square mile utilized this method to compute the lag time given by the relation:

\[
T_{\text{lag}} = 0.6 T_c
\]  

(6)

Where:

- \( T_{\text{lag}} \) = Lag time (hours)
- \( T_c \) = Time of concentration (minutes)

The time of concentration was calculated as:
\[ T_c = T_i + T_t \]  

Where:

\[ T_i = \text{Initial overland flow time (minutes)} \]

\[ T_t = \text{Travel time in a ditch (minutes)} \]

Initial overland flow time was calculated using the formula:

\[ T_i = 1.8 \left( 1.1 - K \right) L_o^{1/2} / S^{1/3} \]  

Where:

\[ K = 0.0132 \times CN - 0.39 \]

\[ CN = \text{Curve number} \]

\[ L_o = \text{Length of overland flow (maximum 500 feet)} \]

\[ S = \text{Average basin slope (\%)} \]

And travel time was calculated as follows:

\[ T_t = \frac{500}{60 V_1} + \frac{(L_t - 500)}{60 V_2} \]  

Where:

\[ L_t = \text{Travel length (ft.)} \]

\[ V_1 = \text{Average Velocity of flow for the first 500 feet of travel distance (ft./sec.)} \]

\[ V_2 = \text{Average Velocity of flow for the second 500 feet of travel distance (ft./sec.)} \]

\[ V_1 \text{ and } V_2 \text{ were calculated as follows:} \]

\[ V_1 = C_1 \left( \frac{S}{100} \right)^{1/2} \]  

Where:

\[ C_1 = 20.2 \text{ for developed areas and 13.8 for undeveloped areas} \]

\[ C_2 = 30.6 \text{ for developed areas and 29.4 for undeveloped areas} \]

\[ S = \text{Average slope for the flow path (\%)} \]
While calculating the time of concentration for urbanized areas, the model considered that the time of concentration calculated using the above did not exceed the time of concentration calculated by the following equation:

\[ T_c = \left( \frac{L}{180} \right) + 10 \]  \hspace{1cm} (13)

Where:

- \( T_c \) = Time of concentration at the first design point in the urban watershed (min.)
- \( L \) = Watershed length (ft.)

### 3.2.1.3 Routing Parameter

The basin model uses the Muskingum and Muskingum-Cunge methods for routing the runoff (CCRFCD, 2002) through the basins. For routing the runoff through the detention basins, the model uses the Modified Puls Method. These methods are described below:

**Muskingum Routing:** Muskingum routing was used to route the runoff through the natural channel, alluvial fans, and sheet flow areas in the watershed. It requires three input parameters: \( X \), \( K \), and \( \text{NSTPS} \).

The \( X \)-parameter accounts for channel or floodplain storage. It was assigned with a value of 0.15 in all the undeveloped areas. The \( K \)-parameter denotes travel time through the routing reach. It was estimated as:

\[ K = \frac{L}{3600 \times V_{\text{wave}}} \]  \hspace{1cm} (14)

Where:

- \( L \) = Length of the routing reach (ft.)
- \( V_{\text{wave}} \) = Wave velocity, assumed to be equal to \( 8/5 \) of average channel velocity
The average channel velocity was calculated using the Manning's equation as follows:

\[ V = 1.49 R^{2/3} S^{1/2} / n \]  

(15)

Where:

- \( V \) = Velocity (ft./sec.)
- \( R \) = Hydraulic radius (ft.), which was assumed as 1.5
- \( S \) = Slope (ft./ft.)
- \( n \) = Manning's roughness coefficient

The NSTPS parameter denotes the number of time steps required. It was taken as the closest integer given by the equation:

\[ \text{NSTPS} = 60 K / \Delta t \]  

(16)

Where:

- \( \Delta t \) = The simulation time steps (min.)

Muskingum-Cunge Routing: Muskingum-Cunge routing was used to route the runoff in improved channels, streets, storm drains, and in the basins where the Muskingum routing yielded unstable results. This method requires the following input parameters:

Manning’s roughness coefficient, base width or diameter (ft.), side slope (xH:1V), energy slope (ft./ft.), and reach length (ft.).

Solution to this method is accomplished by using the following equations:

\[ Q_{i+1}^n = C_1 Q_i^n + C_2 Q_i^{n-1} + C_3 Q_{i+1}^{n-1} + C_4 Q_{i-1}^{n-1} \]  

(17)

The coefficients in the above equation are calculated as:
\[ C_1 = \frac{\Delta t + 2x}{k + 2(1-x)} \]
\[ C_2 = \frac{\Delta t - 2x}{k + 2(1-x)} \]
\[ C_3 = \frac{2(1-x) - \frac{\Delta t}{k}}{k + 2(1-x)} \]
\[ C_4 = \frac{\frac{2(\Delta t)}{k}}{k + 2(1-x)} \]

To determine the above coefficients, \( K \) (travel time through the reach in seconds) and \( X \) (channel or floodplain storage factor) are estimated as:

\[ k = \frac{\Delta x}{c} \]
\[ X = \frac{1}{2} \left( 1 - \frac{Q}{BS_c \Delta x} \right) \]

Where:

- \( Q \) = Discharge (cfs.)
- \( t \) = Time (min.)
- \( x \) = Distance along channel (ft.)
- \( Q_L \) = Lateral Inflow (cfs.)
- \( C \) = Wave Celerity (ft./sec.)

**Modified Puls Routing:** The Modified Puls Routing method was used to route the runoff through the detention basins. This method requires a storage-elevation relationship, an outflow-elevation relationship, and an inflow hydrograph. The relationships, the inflow hydrograph, and a known initial storage condition provide the information necessary to calculate outflow. It relies on a finite difference approximation of the continuity equation and an empirical representation of the momentum equation. In
this method, the Inflow (I), Outflow (D), and storage (S) are related by the following basic equation:

\[(I-D) = \frac{\Delta S}{\Delta t}\]  \hspace{1cm} (18)

Where \(\Delta S\) is the change in storage during the time interval \(\Delta t\). Both \(I\) and \(O\) are time-varying functions with \(I\) and \(D\) being the inflow and outflow hydrographs.

If the average rate of flow during a given time period is equal to the average rate of the flows at the beginning and end of the period, the above equation can be expressed as follows:

\[\frac{(I_1 + I_2)\Delta t}{2} - \frac{(D_1 + D_2)\Delta t}{2} = S_2 - S_1\]  \hspace{1cm} (19)

Where the subscripts 1 and 2 refer to the beginning and end of time period \(\Delta t\).

Rearranging the equation gives the following form used for the Modified Puls method:

\[I_1 + I_2 + \frac{(2S_1}{\Delta t} - D_1) = \frac{(2S_2}{\Delta t} + D_2)\]  \hspace{1cm} (20)

3.3 Steps Involved in Developing the Hydrologic Model

3.3.1 Importing the Basin Model

The first step in developing a model using HEC-HMS is to build the basin model. To create the basin model, the HEC-1 MPU model was imported as shown in Figure 3-8.

This populated the basin model, meteorologic model and control specifications in the project definition window. However, the meteorologic model and control specifications were not needed in this research, so they were deleted.
Figure 3-8. Importing the HEC-1 MPU model to create the basin model for HEC-HMS.

While importing the HEC-1 model, HEC-HMS renamed some elements (reaches, junctions, and subbasins) in the basin model as these names were more commonly used in the HEC-1 MPU model. Often times, the transformation of HEC-1 to HEC-HMS is not consistent. Therefore, after importing the HEC-1 model, the HEC-HMS basin model was opened and each element was verified with the HEC-1 MPU model and the hydrologic map of the watershed. This was done to ensure that the various elements in HEC-1 MPU model were accurately transformed to HEC-HMS and the basin model accurately represented the watershed. A thorough verification found no major difference between the HEC-HMS basin model, HEC-1 MPU model and the watershed map. However, misrepresentations of some of the parameters were found in the basin model. These were detected during the model run and were corrected. The resulting basin model comprised
of 327 basins covering a total area of 216 square miles, 286 junctions, 15 diversion
facilities, 316 reaches, and 11 reservoirs.

The schematic of the entire HEC-HMS basin model is shown in Figure 3-9. A closer
view of the basin model showing the arrangement of different elements is provided in
Figure 3-10.

Figure 3-9. Schematic of HEC-HMS basin model.
3.3.2 Creating the Meteorologic Model

The meteorologic model is a set of information required to define the rainfall to be used in conjunction with a basin model. HEC-HMS provides several options for defining the rainfall in the HMS Meteorologic Model window. This research used the User Gage Weighting (using Thiessen Polygon) method to spatially distribute the precipitation in the watershed (Figure 3-11). This method requires the data for Gages, Subbasin, and Weights. Gages need the data for Gage ID, gage type, total storm depth, and index.
precipitation. Weights need the data for gage ID, gage type, total storm gage weight, and
temporal gage weight for each subbasins. The data for Subbasins need not be entered as it
automatically gets the data once the data for Gages and Weights are entered in the model.

![HEC-HMS meteorologic model](image)

**Figure 3-11.** HEC-HMS meteorologic model

The setting of Gages and Weights in the meteorological model requires processing of
the real-time rainfall data, adding the gages in the model using Hydrologic Engineering
Center-Data Storage System (HEC-DSS), and deriving the factors (weights) for
subbasins to distribute the gage rainfalls using the Thiessen polygon method.
3.3.2.1 Processing of Real-time Rainfall data

CCRFCD has been establishing new gaging stations in the Las Vegas Valley. The last sets of these stations were added in the Flamingo-Tropicana Watershed on June 2004. This research considered all of the available gages for more precise rainfall analysis. Therefore, the rain events having a total rainfall depth of more than 0.5 inches were first identified for the period following June 2004 from the National Weather Service’s rainfall database available for the McCarran International Airport located in the study area. The missing rainfall data for some of the stations were calculated using the arithmetic mean method that involved taking the average of the rainfall for the adjacent neighboring stations.

The beginning of the rainfall was identified based on the changes in rainfall depth (For example, as shown in Table 3-2, the rainfall started at 9:04:25 on 2/28/2004 because the rainfall depth changed from 2.32 inch to 2.52 inch). Since the rainfall varies spatially, the beginning of the rainfall also varied for each gage in the watershed. Therefore rainfall analysis was carried out for each gage in the watershed. The ending of the rainfall was determined based on whether the rainfall completely stopped or did not occur for a six-hour period.

The HEC-HMS model requires time series rainfall data. However, the FTRS gages do not record the rainfall in a fixed time interval (see Table 3-3). Thus, it was necessary to convert the actual rainfall data in to time series.
Table 3-3  Actual rainfall data recorded by the rain gage # 4349.

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>inches</th>
<th>Storm rain</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/30/2004</td>
<td>21:04:24</td>
<td>4.09</td>
<td>1.77</td>
</tr>
<tr>
<td>12/30/2004</td>
<td>9:04:25</td>
<td>4.09</td>
<td>1.77</td>
</tr>
<tr>
<td>12/29/2004</td>
<td>5:23:25</td>
<td>4.02</td>
<td>1.69</td>
</tr>
<tr>
<td>12/29/2004</td>
<td>2:13:30</td>
<td>3.5</td>
<td>1.18</td>
</tr>
<tr>
<td>12/29/2004</td>
<td>1:35:56</td>
<td>3.35</td>
<td>1.02</td>
</tr>
<tr>
<td>12/29/2004</td>
<td>0:48:24</td>
<td>3.19</td>
<td>0.87</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>23:43:25</td>
<td>3.03</td>
<td>0.71</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>21:04:24</td>
<td>2.83</td>
<td>0.51</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>20:15:50</td>
<td>2.83</td>
<td>0.51</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>15:19:08</td>
<td>2.68</td>
<td>0.35</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>14:10:24</td>
<td>2.52</td>
<td>0.2</td>
</tr>
<tr>
<td>12/28/2004</td>
<td>9:04:25</td>
<td>2.32</td>
<td>0</td>
</tr>
<tr>
<td>12/27/2004</td>
<td>9:04:26</td>
<td>2.32</td>
<td>0</td>
</tr>
</tbody>
</table>

This research used 15-minute rainfall data. Therefore, to distribute the rainfall in 15-minute increments, the data was processed using a spreadsheet in two steps as shown in Table 3-4 and Figure 3-12. The first column in the Table 3-3 represents the duration of rainfall with 0:00:000 representing the start time of the rainfall. The second and third column breaks down the duration in the first column to hours and minutes. The fourth column sums the time in column 3 and 4 and presents as total time in minutes. The fifth column provides the accumulated precipitation in inches. At the beginning of the rainfall, the precipitation depth is assigned as zero (if it is not already zero in the actual rainfall data) and is derived on a cumulative basis as the time advances.
Table 3-4  First stage of rainfall data processing (rain gage # 4349).

<table>
<thead>
<tr>
<th>Time</th>
<th>Hour</th>
<th>Minute</th>
<th>Total Time (min.)</th>
<th>Accum P</th>
</tr>
</thead>
<tbody>
<tr>
<td>0:00:00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0:19:27</td>
<td>0</td>
<td>19</td>
<td>19</td>
<td>0.04</td>
</tr>
<tr>
<td>0:35:22</td>
<td>0</td>
<td>35</td>
<td>35</td>
<td>0.08</td>
</tr>
<tr>
<td>0:48:51</td>
<td>0</td>
<td>48</td>
<td>48</td>
<td>0.12</td>
</tr>
<tr>
<td>1:18:55</td>
<td>1</td>
<td>18</td>
<td>78</td>
<td>0.16</td>
</tr>
<tr>
<td>1:21:04</td>
<td>1</td>
<td>21</td>
<td>81</td>
<td>0.2</td>
</tr>
<tr>
<td>1:48:12</td>
<td>1</td>
<td>48</td>
<td>108</td>
<td>0.24</td>
</tr>
<tr>
<td>3:08:37</td>
<td>3</td>
<td>8</td>
<td>188</td>
<td>0.39</td>
</tr>
<tr>
<td>3:33:03</td>
<td>3</td>
<td>33</td>
<td>213</td>
<td>0.43</td>
</tr>
<tr>
<td>4:22:54</td>
<td>4</td>
<td>22</td>
<td>262</td>
<td>0.51</td>
</tr>
<tr>
<td>7:56:18</td>
<td>7</td>
<td>56</td>
<td>476</td>
<td>0.63</td>
</tr>
<tr>
<td>9:09:08</td>
<td>9</td>
<td>9</td>
<td>549</td>
<td>0.67</td>
</tr>
<tr>
<td>9:25:17</td>
<td>9</td>
<td>25</td>
<td>565</td>
<td>0.71</td>
</tr>
<tr>
<td>9:38:40</td>
<td>9</td>
<td>38</td>
<td>578</td>
<td>0.75</td>
</tr>
<tr>
<td>9:53:20</td>
<td>9</td>
<td>53</td>
<td>593</td>
<td>0.79</td>
</tr>
<tr>
<td>10:55:44</td>
<td>10</td>
<td>55</td>
<td>655</td>
<td>0.83</td>
</tr>
<tr>
<td>11:23:42</td>
<td>11</td>
<td>23</td>
<td>683</td>
<td>0.9</td>
</tr>
<tr>
<td>11:49:04</td>
<td>11</td>
<td>49</td>
<td>709</td>
<td>0.94</td>
</tr>
<tr>
<td>12:04:55</td>
<td>12</td>
<td>4</td>
<td>724</td>
<td>0.98</td>
</tr>
<tr>
<td>12:13:10</td>
<td>12</td>
<td>13</td>
<td>733</td>
<td>1.02</td>
</tr>
<tr>
<td>12:41:36</td>
<td>12</td>
<td>41</td>
<td>761</td>
<td>1.18</td>
</tr>
<tr>
<td>12:46:56</td>
<td>12</td>
<td>46</td>
<td>766</td>
<td>1.22</td>
</tr>
<tr>
<td>13:14:42</td>
<td>13</td>
<td>14</td>
<td>794</td>
<td>1.34</td>
</tr>
<tr>
<td>13:30:28</td>
<td>13</td>
<td>29</td>
<td>809</td>
<td>1.46</td>
</tr>
<tr>
<td>14:14:38</td>
<td>14</td>
<td>14</td>
<td>854</td>
<td>1.5</td>
</tr>
<tr>
<td>15:09:34</td>
<td>15</td>
<td>9</td>
<td>909</td>
<td>1.57</td>
</tr>
<tr>
<td>15:16:55</td>
<td>15</td>
<td>16</td>
<td>916</td>
<td>1.61</td>
</tr>
<tr>
<td>15:57:32</td>
<td>15</td>
<td>57</td>
<td>957</td>
<td>1.73</td>
</tr>
<tr>
<td>16:16:56</td>
<td>16</td>
<td>16</td>
<td>976</td>
<td>1.85</td>
</tr>
</tbody>
</table>

After completing the above procedure, the rainfall is distributed in 15-minute time intervals for all the gaging stations in the watershed. This was done using the VLOOKUP function in MS Excel as shown in Figure 3-12.
Figure 3-12. Distributing the rainfall in 15-minute time interval using the VLOOKUP function in MS Excel.

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3.3.2.2 HEC-Data Storage System

After distributing the rainfall in 15-minute time intervals, it was required to input these data into the model. HEC-HMS provides an efficient way to input the precipitation information into the model using the external data storage system. The 15-minute rainfall data was converted to the HEC-DSS using the Data Exchange Add-In software developed by USACE. The software is free to download and is used with Microsoft Excel.

Table 3-5 The required DSS format of rainfall data.

<table>
<thead>
<tr>
<th>Part A:</th>
<th>FLAMINGO TROPICANA WATERSHED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part B:</td>
<td>STATION 4274</td>
</tr>
<tr>
<td>Part C:</td>
<td>PRECIP-CUM</td>
</tr>
<tr>
<td>Part D:</td>
<td>28DEC2004</td>
</tr>
<tr>
<td>Part E:</td>
<td>15MIN</td>
</tr>
<tr>
<td>Part F:</td>
<td>FTRS</td>
</tr>
<tr>
<td>Beg. Date:</td>
<td>28-Dec-04</td>
</tr>
<tr>
<td>Beg. Time:</td>
<td>930</td>
</tr>
<tr>
<td>End Date:</td>
<td>29-Dec-04</td>
</tr>
<tr>
<td>End Time:</td>
<td>1945</td>
</tr>
<tr>
<td>Units:</td>
<td>IN</td>
</tr>
<tr>
<td>Data Type:</td>
<td>Index INST-CUM</td>
</tr>
<tr>
<td>12/28/2004 9:30 0.00</td>
<td></td>
</tr>
<tr>
<td>12/28/2004 9:45 0.00</td>
<td></td>
</tr>
<tr>
<td>12/28/2004 10:00 0.00</td>
<td></td>
</tr>
<tr>
<td>12/29/2004 19:15 2.40</td>
<td></td>
</tr>
<tr>
<td>12/29/2004 19:30 2.40</td>
<td></td>
</tr>
<tr>
<td>12/29/2004 19:45 2.40</td>
<td></td>
</tr>
</tbody>
</table>

To develop the HEC-DSS, the rainfall data are required to be converted into DSS format. HEC-HMS is very sensitive in reading the data type used in the HEC-DSS. Therefore, care must be taken to accurately produce the data into DSS format, as any
flaws in the formatting results in inaccurate model results. The DSS format used in this research is shown in Table 3-5.

Using the format shown above, the precipitation data for all the 32 gaging stations were converted into HEC-DSS, which contained individual gages. Theses gages were then added in the model as shown in Figure 3-13.

![HEC-DSS method to create the gages in the HEC-HMS.](image)

**Figure 3-13.** HEC-DSS method to create the gages in the HEC-HMS.

### 3.3.2.3 Distributing the Rainfall in Basins

To distribute the rainfall in the basins, Thiessen polygons were drawn using the built in feature in ArcGIS 9. The Thiessen polygon provides a way to determine the relative weight of each gage within a subbasin. To make the Thiessen polygon, first, the feature data of the gaging stations (point data) was converted to point coverage. Then, using the
point coverage of the gaging stations and the watershed polygons, the Thiessen polygon were drawn as shown in Figure 3-14.

In the Thiessen diagram, each polygon contains a basin or a portion of the basin. Where polygon boundaries contained an entire basin, a weight of 1 was assigned, which means 100% of the basin precipitation is applied to that gage. Intersected basins were assigned weights depending on the percentage of the basin area that each gage contributes to. For example, Figure 3-15 shows the weights for subbasin FW37 located at the upper northwest region in the watershed (see Figure 3-14). The Thiessen polygon line divides this subbasin into two indicating that the precipitation from 70% area of this subbasin is applied to the gage station # 4084 and 30% to # 4394.

![Figure 3-15. Specifying the Thiessen weights in the precipitation model.](image)

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3.3.3 Setting Control Specifications

The control specifications define time-related information for a simulation. There are specific formats for entering the time, date and time interval in the control specifications for simulating runoff. The format of the date and time data used is shown in Figure 3-16.

![HEC-HMS control specifications set-up window.](image)

In this research, the control specifications were set to start the computation of hydrograph with the start of the rainfall. It was continued till the ordinate of the hydrograph completely recessed using a 15-minutes time interval for computation.
3.4 Model Run Using Historic Rain Events

After setting the control specifications, a simulation run was created. To create a run, a Basin ID, Met Model ID, and Control ID need to be specified as shown in Figure 3-17.

![HMS * Run Configuration](image)

Figure 3-17. Putting together the basin model, meteorologic model, and control specifications to create a run for model simulation.

The run created above is kept under the run manager as shown in Figure 3-18. The run is to be selected for computing the simulation.
Figure 3-18. The run created using the run configuration is stored in run manager.

When the model was executed for the first time, the computation did not complete. The model generated errors, warnings and notes. These errors were reviewed and addressed to complete the model run.

The errors were mostly associated with the basin model. The description of the various errors produced while running the model and the ways these errors were addressed are described in the following paragraphs.

The first sets of errors were concerned with the SCS curve number in the loss model. Some of the basins in the imported basin model contained missing and some had invalid curve numbers. The curve numbers were entered and corrected by referring back to the HEC-1 basin model. This also necessitated a thorough verification of all the parameters in the new HMS basin model to confirm that the various parameters in the imported basin model were the same as that of the HEC-1 basin model.

The second kind of error was related to the invalid storage-outflow table of the reservoirs. HEC-HMS uses the storage-outflow table to calculate the outflow using the
interpolation technique. Since the table was invalid (figure in the table were duplicated), HMS could not calculate the outflow and the storage-outflow table was corrected to allow for interpolation/computation.

The next type of errors read "Root is not bracketed in equation sover: Brent's method". This error message means that the computed storage in the reservoir exceeds the maximum storage in the elevation-storage curve used by the reservoir. HEC-HMS does not extrapolate. To correct this kind of error, the elevation-storage table was extended with additional data.

The last set of errors in the basin model was about the missing side-slope data for certain reaches. In HEC-1, it is common practice to leave a blank instead of assigning zero value for side-slope when the value for water depth in the reach is non-zero. But HEC-HMS does not read blanks. Hence, the reaches with missing side slopes were assigned a zero value.

Once the above errors were addressed, the simulation was carried out. The computation results can be viewed in the basin model by right clicking the hydrologic elements in the basin with the mouse and selecting the desired options in the resulting window. HEC-HMS provides different options to view the computed results. They include graphs, summary tables, and a time-series table with information on peak flow, time to peak, and total volume. A careful observation of the basin data revealed that the distributions of precipitation in the basins were very low. To correct this, a rigorous review of the entire model was carried out and it was found that the problem was within the HEC-DSS. The data type in the DSS file was not correct. To correct this problem, the data type in DSS was corrected as per Table 3-4.
Most of the above errors in HEC-HMS version 2.2.2 are due to bugs. USACE is continuously working to remove these pitfalls in HMS. As a result, these bugs have been corrected in the newer version of HEC-HMS 3.0.1 (Fleming, 2006).

After correcting the above errors, the HEC-HMS was run successfully. The computation result for each element in the basin model was observed and no discrepancy was found in the results. The observed gage at the outlet point was also created in the model, in the same way as was done to create the gages, to compare it with the simulated flow. Further, the model was run for two more rainfall events in the watershed. The model results and description of rainfall events are provided in Chapter 4.

3.5 Sensitivity Analysis (Optimization)

Oftentimes, in rainfall-runoff models, the simulated runoff is different from the observed runoff. This is due to the fact that the gage-recorded rainfall data are associated with a lot of uncertainties and models are an estimate of real hydrologic processes. These uncertainties arise from the rainfall recording and are further compounded as the data are processed to derive hydrological information in a format suitable for inputting the data into the meteorologic model. Moreover, the uncertainties also come from oversimplification of the model itself and from the parameters used in the model (Nagai, 2002). To overcome this problem, the parameters for the different methods included in the subbasin and reach elements need to be optimized. Optimization estimates new parameters that, if included in the model, generate the new runoff that is close to the observed runoff. However, optimization requires observed runoff for at least one element
in the model in order to estimate the new parameter upstream of that element. The results of parameter optimization are included in Chapter 4.

3.6 Modeling with Radar Rainfall Data

Hydrologic modeling with radar rainfall data was another attempt to develop a rainfall-runoff model for flood forecasting. This research used two different sets of radar rainfall data. Radar data with 1 KM spatial resolution had a temporal resolution of 5 minutes and 2 KM spatial resolution had a temporal resolution of 15 minutes. The radar rainfall data were obtained from OneRain, Inc., a commercial rainfall and environmental data vendors of Colorado. The time series radar rainfall data were adjusted based on the gage rainfall data and was derived for each individual subbasins (i.e. basin averaged) of the Flamingo Tropicana watershed.

The overall steps involved in developing this hydrologic model for radar data was the same as explained in section 3.3 for gage rainfall. However, the meteorologic model used different methods to distribute the precipitation in the watershed. A brief description of the modeling procedure involved is provided below.

3.6.1 Model Development

To develop the model using radar data, a new HEC-HMS project was opened and the basin model that was developed using the HEC-1 MPU model for gage rainfall data was imported into the HEC-HMS model. As described in section 3.3.1, the “import” function in the HEC-HMS caused the basin model to lose/modify some of its parameters. Hence, the resulting basin model was thoroughly verified with the original basin model used with the gage rainfall data. The reservoirs were missing required data. Therefore, all the reservoirs were reassigned with their storage-outflow parameters.
The sensitivity analysis provided a value for the initial loss. After reviewing the basin model for consistency, the initial loss parameters obtained from sensitivity analysis were entered for each subbasin in the basin model. As described in section 3.2.1.1, the basin model used Initial Constant and SCS-CN method for loss prediction. Since, the undeveloped basins in the watershed were assumed to remain undeveloped and the USACE have already established initial loss parameters for these basins, the initial loss (in.) parameters were assigned only to the developed basins. 296 out of 327 subbasins were assigned with this new parameter as shown in Figure 3-19.

![Table](image)

**Figure 3-19.** New basin model that includes the initial loss (0.5 inch) parameter obtained by sensitivity analysis.
Next, the meteorologic model was developed. The model used 1 KM and 2 KM resolution rainfall data of 28-29 December 2004. Since the gridded radar data was derived for each subbasin (i.e. basin-averaged), the meteorologic model developed for gage precipitation data using the Thiessen Polygon method was not applicable for radar data. Hence, a new meteorologic model was created. The model used the User Hyetograph method to distribute the rainfall in the watershed.

Prior to developing the meteorologic model, the gages were installed in the HEC-HMS model from the HMS Project Definition window. The gages were added in the model using the HEC-DSS method as described in section 3.3.2.2. Since the radar data was basin-averaged, each basin represented an individual gage and hence the model contained a total of 327 gages.

![HEC-HMS window to add the gages in the model.](image)

Figure 3-20. HEC-HMS window to add the gages in the model.
Besides the gages, the meteorologic model also needs the subbasins. Therefore the subbasins were added in the meteorologic model using the subbasin tab. Once the subbasins were entered, the gages were assigned to the respective subbasins using the drop down menu in "Gage" ID column of the meteorologic model. The resulting meteorologic model using the User Hyetograph option is shown in the Figure 3-21.

![Figure 3-21. Meteorologic model used for radar rainfall data using User Hyetograph Method.](image)

Finally, the control specification was created as described in section 3.3.3 using the 15 minute time interval for hydrograph computation. The result of the radar model is presented in section 4.4.
CHAPTER 4

RESULTS

This chapter presents the results of the hydrologic modeling beginning with the rainfall analysis. The hydrologic model was first tested with historical precipitation events. Sensitivity analysis was performed to evaluate the optimum parameter for the model. Finally, the model was run with gage-adjusted radar rainfall data. The results for the Flamingo-Tropicana Watershed are presented below.

4.1 Rainfall Analysis

The research used real-time rainfall data available through the FTRS of CCRFCD. As discussed in Chapter 3, the rainfall data was processed to distribute it in 15-minute time intervals. An example of actual and estimated rainfall for is shown in Figure 4-1 (see Appendix 3 for actual and estimated rainfall data for all the gages used in the model). The actual rainfall shows light precipitation at the beginning of storm, intense precipitation during the middle, and decreasing precipitation at the end. The estimated rainfall preserves this pattern of the actual precipitation, while being distributed in equal time intervals of 15-minutes.
Figure 4-1. Actual real-time rainfall and the estimated rainfall of 15-minutes time interval for December 28-29, 2004 rainfall event for the gage # 4399.

The hydrologic model developed in this research was run and tested with three historical rainfall events: December 28-29, 2004, November 21-22, 2004, and July 24, 2005. Figure 4-2 compares these rainfall events. The general pattern is the same for all the three precipitation events. However, the total depths of the precipitation are different. The December storm was a large storm with higher rainfall depth of all the three storms and it produced 1.86 inches of rainfall in the watershed, the November storm produced 0.81 inches of rainfall, and the July storm was a smaller storm that produced a rainfall depth of 0.58 inches.
Figure 4-2. Comparison of the three precipitation events used in the study.

4.2 Model Results for Gage Rainfall Data (Uncalibrated)

The various results of the model based on gage rainfall data are described in the following sections. First, example results are provided for the different elements of the watershed, which is followed by the results from all the rainfall events.

4.2.1 Examples of Results for Individual Basin Elements

The HEC-HMS model computes the results for each of the watershed elements used in the basin model. These results are produced in the form of a graph, summary table, and time-series table and can be viewed by right clicking the elements and choosing the desired form of result shown in the drop down window. The results produced in the time-series format can be retrieved using HEC-DSS for further analysis by the user. The results in the summary table can be viewed either in inches or acre-Feet. This research
used inches for the computation, as shown in the summary result window for different basin elements. The model results for different basin elements in the form of a summary table and graph, based on the storm events of 28-29 December 2004, are described below.

4.2.1.1 Subbasin

The subbasin is used to represent the individual basin in the watershed. The subbasin result provides information on: (1) peak discharge from the subbasin, (2) date and time of peak discharge from subbasin, (3) total precipitation received by the subbasin, (4) total loss in the subbasin, (5) total excess precipitation produced by the subbasin, (6) total base flow in the subbasin, and (7) total discharge from the subbasin. These values for subbasin RD3 are shown in the summary results in Figure 4-3.

Figure 4-3. Subbasin result in the form of summary table.
It should be noted that the total precipitation falling in the basin is equal to the total loss and total excess precipitation. Figure 4-4 provides the result in graphical form for the same subbasin, and storm.

![Graphical representation of subbasin result](image)

Figure 4-4. Subbasin result in the form of graph.

4.2.1.2 Junction

The junction represents river or stream confluence and has one or more inflow and one outflow. The junction provides computed results for: (1) peak outflow from the junction, (2) date and time of peak outflow from the junction, and (3) total outflow from the junction. Figure 4-5 shows these values for the junction CRD3.
Figure 4-5. Junction result in the form of summary table.

Figure 4-6 provides the graphical representation of the results for the above junction. In the graph, the top line represents the outflow from the junction, which is the sum of all the other lines below it that represent the inflows to the junction.
4.2.1.3 Reservoir

The reservoir denotes the detention basins in the watershed. It may have more than one inflow but only one computed outflow. HEC-HMS model computes outflow based on the storage-outflow relationship of the reservoir. The reservoir result provides the information about: (1) peak inflow to the reservoir, (2) date and time of peak inflow to the reservoir, (3) peak outflow from the reservoir, (4) date and time of peak outflow from the reservoir, (5) total inflow to the reservoir, (6) total outflow from the reservoir, (7) peak storage in the reservoir, and (8) peak elevation in the reservoir (as shown in Figure 4-7 for reservoir RRDB).

![HMS * Summary of Results for Reservoir RRDB](image)

Figure 4-7. Reservoir result in the form of summary table.
Figure 4-8 provides the graphical representation of the reservoir results. It shows that the inflow to the reservoir is fluctuating, while outflow from it is constant and controlled.

Figure 4-8.  Reservoir result in the form of graph.

4.2.1.4 Reach

The reach is an element with one or more inflows and only one outflow. The reach result provides the information on: (1) peak inflow to the reach, (2) date and time of peak inflow to the reach, (3) peak outflow from the reach, (4) date and time of peak outflow from the reach, (5) total inflow to the reach, and (6) total outflow from the reach as seen in Figure 4-9 for the reach RCRD2.
Figure 4-9. Reach result in the form of summary table.

Figure 4-10 provides the graphical representation of the reach results. The two lines in the graph represent the amount of inflow and outflow to and from the reach. Inflow and outflow are the same for the reach as the losses in the reach are very small.
4.2.1.5 Diversion

The diversion is an element with two outflows, one diverted flow and one main flow, and one or more inflows. It provides the result for: (1) peak inflow to the diversion, (2) date and time of peak inflow to the diversion, (3) peak outflow from the diversion, (4) date and time of peak outflow from the diversion, (5) peak diversion, (6) date and time of peak diversion, (7) total inflow to the diversion, (8) total outflow from the diversion, and (9) total diversion. The result for the diversion PASSTMDB is shown in Figure 4-11. It shows that the total inflow to the diversion is equal to total outflow and total diversion.
Figure 4-11. Diversion result in the form of a summary table.

Figure 4-12 provides the graphical representation of the diversion results. The figures show that the diversion facility is diverting more than 95% of the inflow.
4.2.2 Results from the Various Storm Events

The results presented below for the three storm events are based on the comparison of the model flow with the observed flow at the outlet of the watershed. The model flow was obtained from the junction CFW38 at basin FW38, which represents the outlet of the watershed; and the corresponding observed flow data was obtained from the USGS Gage # 094196781 located at Flamingo Wash in Nellis Boulevard (see Figure 3-5).
4.2.2.1 Storm Event 1: December 28-29, 2004

The December 28-29, 2004 event was a large winter storm. It produced light rainfall in the beginning, which was followed by heavy rainfall. Figure 4-13 shows a graph of the basin average rainfall intensity vs. time for this storm event. The rain started at 09:30 on December 28th and lasted till 19:45 on December 29th. However, the rain was considerably less after 09:45 on December 29th. The rainfall produced highest rainfall intensity of 0.37 in/hr, which was recorded at 04:30 on December 29th.

![Graph of basin average rainfall intensity vs. time for December 28-29, 2004 rainfall.](image)

Figure 4-13. A plot of basin average rainfall intensity and time for the December 28-29, 2004 rainfall.

This storm resulted in an average of 1.86 inches of rain over the basin. The highest total rainfall depth recorded was 2.4 inches at gage # 4374 and the lowest was 0.98 inches.
at gage # 4344 as shown in Figure 4-14. Eleven out of 32 rain gages recorded more than 2.0 inches of rainfall in the watershed.

Figure 4-14. Isohyet of total rainfall depth for December 28-29, 2004 storm.

Figure 4-15 shows the model result for this storm event along with the observed flow recorded by USGS at the outlet of the watershed. The plot reveals that though the model flow is over-estimated compared to the observed flow, the timing of the two hydrograph peaks matches well. For example, the simulated peak flow was 4022 cfs occurred at 05:35 on December 29th. On the other hand, the observed peak was 2530 cfs that occurred at 05:30 on December 29th. This showed that the time to peak for simulated flow compares well with the time to peak for the observed flow.
Figure 4-15. Comparison of the computed and observed hydrograph for December 28-29 storm.

The rising and recession limbs of the two hydrographs also show an adequate match, however, there is more flow in the recession limb of the model hydrograph. The comparison of the observed and simulated peaks shown in Figure 4-15 with rainfall peak shown in Figure 4-13 also revealed that these peaks occurred after the peak rainfall depth that was recorded at 04:30 on December 29th.

The comparison of the observed and the computed hydrograph indicated that the model generally represented the overall shape of the hydrograph reasonably well, and the model provided an excellent prediction of the time to peak.
**Summary of Results for Junction CFW38**

- **Project:** Flamingo Tropicana
- **Run Name:** Run 1
- **Junction:** CFW38
- **Start of Run:** 28 Dec 04 0930
- **End of Run:** 29 Dec 04 2400
- **Basin Model:** Flamingo Tropicana
- **Met. Model:** Ram Trop Dec 2004
- **Execution Time:** 04 Jul 06 1711
- **Control Specs:** Flam Trop Dec 2004
- **Volume Units:** Inches/Acre-Feet

**Computed Results**

- Peak Outflow: 4021.5 (cfs)
- Date/Time of Peak Outflow: 29 Dec 04 0545
- Total Outflow: 0.21 (in)

**Observed Hydrograph at Gage: FW38**

- Peak Discharge: 2530.0 (cfs)
- Date/Time of Peak Discharge: 29 Dec 04 0530
- Avg. Abs. Residual: (cfs)
- Total Residual: (in)
- Total Obs. Discharge: 0.16 (in)

---

Figure 4-16. Summary of results for computed and observed flow for Flamingo Wash at Nellis Boulevard.

### 4.2.2.2 Storm Event 2: November 21-22, 2004

The November 21-22, 2004 was also a winter storm event. The storm produced higher rainfall in the beginning followed by gradually receding rainfall. A graph of basin average rainfall intensity with time is provided in Figure 4-17. The rain started at 07:30 on November 21\(^{st}\) and lasted until 16:45 on November 22\(^{nd}\). The highest rainfall intensity of 0.17 in/hr was recorded at 18:00 on November 21\(^{st}\).
Figure 4-17. A plot of basin average rainfall intensity and time for the November 21-22, 2004 rainfall.

This storm produced an average of 0.81 inches of rainfall in the study area. The highest total rainfall depth was 1.11 inches recorded by the gage # 4374, and the lowest was 0.07 inches at gage # 4084 as shown in Figure 4-18. Out of 32 rain gages in the watershed, 29 rain gages recorded more than 0.50 inches of rainfall in the watershed.
Figure 4-18. Isohyet of total rainfall depth for November 21-22, 2004 storm.

Figure 4-19 shows the model result of this storm event. The observed hydrograph was also introduced in the model to compare the model flow with observed flow. For this storm event, it was observed that the model peak flow was less than the observed peak flow. The model produced two peaks. These peak flows were produced after the peak rainfall depth as shown in Figure 4-17. The highest peak flow of 743 cfs occurred at 07:00 on November 22\textsuperscript{nd}, and the next highest peak of 727 cfs produced at 20:45 on November 21\textsuperscript{st}. The observed hydrograph also had two peaks. The first peak was noticed at 20:50 on November 21\textsuperscript{st}. The model appears to simulate the time to peak very closely as the timing of the first peak for the observed hydrograph was closely in agreement with the timing of the first peak of the model hydrograph.
4.2.2.3 Storm Event 3: July 24, 2005

The third storm was a summer event on July 24, 2005. Figure 4-20 shows a graph of the basin average rainfall intensity and time for this storm event. The rain started at 00:30 on July 24th and continued till 13:00 of the same day. The highest rainfall intensity of the storm was 0.27 in/hr occurring at 04:45.
Figure 4-20. A plot of basin average rainfall intensity and time for the July 24, 2005 rainfall.

This storm produced an average of 0.58 inches of rainfall over the watershed. The highest rainfall depth of 1.22 inches was recorded by the gage # 4334 and the lowest of 0.19 inches was recorded at gage # 4449 as shown in Figure 4-21. Nineteen out of 32 rain gages recorded more than 0.5 inches of rainfall in the watershed.
The simulated hydrograph of this storm event is shown in the Figure 4-22. The observed hydrograph is also included in the model. Similar to the other storms, the simulated peak flow underestimated the observed peak flow. The model produced two peaks. Both of these peaks were produced after the peak rainfall depth. The highest peak flow of 570 cfs occurred at 09:00 on July 24th and the next highest peak of 542 cuffs was produced at 06:30 on July 24th. The observed hydrograph had one peak occurring at 06:25 on July 24th. The timing of this first peak for the observed hydrograph was in close agreement with the timing of the first highest peak of the simulated hydrograph. As with the November 21-22, 2004 storm, the model appears to simulate the time to peak fairly well.
Figure 4-22. Comparison of the computed and observed hydrograph for July 24, 2005 storm.

4.2.3 Overall Performance

The overall model performance was evaluated by comparing the simulated peak flow with the observed peak flow at three locations shown in the Figure 4-23: Upper Flamingo Detention Basin (UPFLDB), Tropicana Detention Basin (TRDB), and Flamingo Wash at Nellis Boulevard (FWNB). Figure 4023 also shows the path of runoff in the watershed as it flows from east to west.
The UPFLDB is located at the upper region of the study area and receives runoff from 90 square miles of tributary through the Flamingo Wash. The TRDB is located about 3.5 miles downstream of the UPFLDB. TRDB receives flow from UPFLDB and also the southwest region of the study area through Tropicana Wash. The total drainage area of the TRDB is 175 square miles. Finally, the flow from TRDB continues to the east and exits the watershed at Nellis Boulevard with 216 square miles of total tributary.

The simulated peak flows at these three locations were obtained by running the model and selecting (right clicking the mouse) the reservoirs UPFLDB and TRDB. For FWNB, the simulated peak flow was found from the main outlet of the watershed (selecting...
junction CFW38 below basin FW38 shown in Figure 3-5). The summary table provides peak flow data. The observed peak flow for the gages was calculated using the stage data as described in section 3.1.2. The comparisons of the peak flow for different storms are provided below.

December 28-29, 2004 Storm: The comparison of observed flow and model flow for this storm is shown in the Table 4-1. The model simulated peak flow is greater than the observed peak flow at all the three locations. However, the times to peak for the observed flow and the simulated flow are in good agreement. The time to peak for both the flows is the same (21:00 hours) at UPFLDB. Peak times vary slightly as the flow proceeds downstream to TRDB and FWNB. In both the cases, the model-generated time to peak is earlier than the time to peak for observed flow. The model time to peak is 45 minutes earlier at TRDB and at FWNB it is 35 minutes before.

Table 4-1 Flow comparison for December 28-29, 2004 storm.

<table>
<thead>
<tr>
<th>Location</th>
<th>Observed</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Flow (cfs)</td>
<td>Time to Peak (hr)</td>
</tr>
<tr>
<td>UPFLDB</td>
<td>138</td>
<td>21:00</td>
</tr>
<tr>
<td>TRDB</td>
<td>104</td>
<td>22:25</td>
</tr>
<tr>
<td>FWNB</td>
<td>2530</td>
<td>20:50</td>
</tr>
</tbody>
</table>

July 24, 2005 Storm: The comparison of flow for the July 24, 2005 storm is shown in the Table 4-2. At UPFLDB and TRDB, the model predicted peak flow is more than the observed peak flow. However, the model predicted peak flow is less than the observed peak flow at FWNB. When the time to peak is compared for observed and model flows, it
is observed that they are closely matching. Both the flow peaked at the same time at
UPFLDB. Whereas, the model peak flow is 5 minutes earlier at TRDB and at FWNB it
was 5 minutes later.

Table 4-2  Flow comparison for July 24, 2005 storm.

| Location | Observed | | Model |
|----------|----------|----------|
|          | Peak Flow (cfs) | Time to Peak (hr) | Peak Flow (cfs) | Time to Peak (hr) |
| UPFLDB   | 13       | 6:10      | 148        | 6:00      |
| TRDB     | 18       | 5:35      | 266        | 6:00      |
| FWNB     | 3600     | 5:40      | 542        | 6:30      |

November 21-23, 2004 Storm: Table 4-3 provides a comparison of flow for this
storm. The model simulated peak flow is greater than the observed peak flow at UPFLDB
and TRDB. Whereas, it is much lower at the FWNB. However, the times to peak for the
observed flow and the model simulated flow are in good agreement with each other. The
simulated time to peak at UPFLDB was 10 minutes earlier than the observed time to
peak. At TRDB and FWNB, it was 25 and 50 minutes later than the observed flow.

Table 4-3  Flow comparison for November 21-23, 2004 storm.

| Location | Observed | | Model |
|----------|----------|----------|
|          | Peak Flow (cfs) | Time to Peak (hr) | Peak Flow (cfs) | Time to Peak (hr) |
| UPFLDB   | 11       | 13:30     | 105        | 13:30     |
| TRDB     | 15       | 12:15     | 286        | 12:10     |
| FWNB     | 1880     | 13:40     | 727        | 13:45     |
4.2.4 Summary of Uncalibrated Model Results

The comparison of the flows at the three locations for the three storms revealed that while the model successfully predicted the time to peak, it could not predict the peak flow well. The model simulated flow was more than the observed flow at UPFLDB and TRDB, whereas, it was lower than the observed flow at FWNB (outlet of the watershed). Overall, the model overestimated the observed peak flow for the December 2004 storm; whereas, it underestimated the observed peak flow for November 2004 and July 2005 storm.

The total flow volume for the model and observed flow was also compared as shown in the Table 4-4. It is noticed that the model flow is more for the December 2004 storm whereas it is less for other two storms.

<table>
<thead>
<tr>
<th>Storm</th>
<th>Observed Flow (Acre-Feet)</th>
<th>Model Flow (Acre-Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28-29 December 2004</td>
<td>1915.5</td>
<td>2494.3</td>
</tr>
<tr>
<td>21-23 November 2004</td>
<td>939.0</td>
<td>639.1</td>
</tr>
<tr>
<td>24 July 2005</td>
<td>560.0</td>
<td>442.56</td>
</tr>
</tbody>
</table>

The difference in the volume of simulated flow and observed flow for the December 2004 storm led to the conclusion that the parameters need to be optimized in the model in order to achieve the best fit between the two flows.
4.3 Parameter Optimization

The results from the previous sections indicate that there exists a need for calibration. Calibration is generally done with respect to the observed data in combination with sensitivity analysis, and is used to identify the parameter values that enable the best possible fit of the computed and observed hydrographs. While identifying the parameter values, the impact on peak flow rate, the time to peak and the overall hydrograph shape were examined.

The model was used to simulate the runoff for three different storm events in the Flamingo-Tropicana watershed. As discussed in the previous sections, the model underestimated the peak flow for the November 2004 and July 2005 storm as these storm event were minor compared to December 2004 storm. Hence, these storms were not used for further analysis. Therefore, the sensitivity analysis was performed using the storm event of 28-29 December 2004 and stream flow data for the same duration from the USGS Flow Gage # 094196781 located at Nellis Blvd. The location of the USGS gage is illustrated in Figure 3-5. Figure 4-16 shows the peak flow and time to peak for model flow and observed flow. The time of peak computed by the model was 05:45 AM on 29th December 2004. This compared well with the observed time of peak of 05:30 AM on 29th December 2004. Realizing that the computation time interval of the control model was in 15 minute steps, the model can be said to provide a fairly accurate estimation of time to peak.

However, the model peak flow was about 59% higher than the observed peak flow. This is because the model represented the ultimate built-out condition in the watershed with all planned flood control facilities in place. The model therefore had very low
precipitation loss. However, the watershed was in fact not fully built up as envisioned in the model (see figure 3-6) and there were more losses in the watershed before the runoff began. These losses could be the water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration that generally depend on the soil and cover parameters. Since the watershed was not fully developed, the infiltration, interception and storage losses were not accurately accounted by the model, and hence the model peak flow was higher than the observed peak flow.

To lower the model flow and to best fit the model peak flow with the observed peak flow, a series of sensitivity analyses were performed by varying the initial loss (in.) parameter in the basin model for the 180 square miles of developed basins downstream of the Upper Flamingo Wash Detention Basin. The loss model used the SCS-CN method to compute the loss. This method generally requires the curve number, initial loss (in.), and percent impervious factor. Initially, the model used curve number values for each basins based on ultimate developed scenario. The initial losses were assigned zero (but by default the initial loss is computed as 0.2 times of the potential maximum retention of the soil (S), where $S = \frac{1000}{CN-10}$) and the % imperviousness were also assigned zero because the curve numbers already accounted for this factor. Hence, to lower the model peak flow, only the initial loss values were suitable for varying. Therefore, new initial losses were assigned to each subbasins and the model was simulated in each trial until the desired peak flow was obtained considering the overall shape of the hydrograph. The initial loss was increased by 0.1 inch per trial in seven trials. Figure 4-24 shows the results obtained from each trial. It was observed that the lowest possible peak flow

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generated by the model with an initial loss value of 0.7 inch was 3230 cfs. The time to peak also remained unchanged until the initial loss value was 0.7 inch.

![Graph showing computed and observed flows against initial loss](image)

Figure 4-24. Result of the sensitivity analysis performed to lower the computed peak flow close to observed peak flow by increasing the initial loss in the basin model.

However, the overall shape of the hydrograph obtained with an initial loss of 0.7 inch did not match satisfactorily with the observed hydrograph. A closer analysis of the hydrographs produced by using different values of initial loss revealed that a value of 0.5 inch produced a hydrograph that adequately matched with observed hydrograph. The comparison of model flow with initial loss of 0.5 inch and observed flow is shown in Figure 4-25.
Figure 4-25. Comparison the observed flow and model flow after incorporating the result of sensitivity analysis (i.e. with initial loss value of 0.5 inch) in the model.

4.4 Modeling with the Radar Rainfall Data

Sensitivity analysis provided a value of 0.5 inch for the initial loss. This value was included in the model to represent the existing basin condition. Since the uncalibrated model with gage precipitation data underestimated observed flow for July 21, 2005 and November 21-23, 2004 storm, these storms were considered inappropriate for further modeling with radar data. Hence, the radar data for December 28-29, 2004 storm was acquired for modeling.
4.4.1 Description of the Radar Rainfall Data

The study used 1 KM and 2 KM resolution gridded rainfall data. The radar data were adjusted/calibrated with the observed gage rainfall data for more accurate rainfall values. The western region of the watershed has no gages (see Figure 3-4). Therefore the radar data used larger domain for radar data calibration/adjustment and the gages outside of watershed at the western region were also used to derive more accurate rainfall values for western regions in the watershed. The gridded rainfall data were distributed to each subbasin in the watershed using the gage/radar ratio from the nearest five gage locations and the kriging based interpolation scheme was used to assign rainfall values to each subbasin.

Figure 4-26 provides the choropleth map of total rainfall depth derived from 1 KM resolution radar data for each subbasin in the watershed.

The rainfall intensity of radar rainfall along with gage rainfall is shown in Figure 4-27. The general pattern is the same for all the three rainfalls. As seen in the figure, the radar data produced a highest rainfall intensity of 0.39 inches/hour in the watershed but for other minor peaks, the intensity is mixed in the sense that sometime the radar data produced higher intensity and sometimes the gage. However, the timing of rainfall intensities for individual peaks for each of the rainfall match very well throughout the rainfall period.

The average rainfall depth produced by the radar data is higher than the gage rainfall data. 1 KM and 2 KM radar data produced 2.00 and 1.96 inches of rainfall depth respectively in the watershed, whereas; the gage rainfall produced 1.86 inches.
Figure 4.26. Choropleth map of 1 km resolution basin average gridded rainfall data.
4.4.2 Model Results

The modeling was carried out with radar data with a computation interval of fifteen minutes for hydrograph generation. The hydrographs generated with 1 KM and 2 KM radar rainfall data were compared graphically with the hydrograph generated with gage rainfall data at various locations in the watershed. As shown in Figure 4-23, these locations were the same locations that were used to compare the results of uncalibrated model with observed flow. However, instead of using the stage data from Upper Flamingo Detention Basin and Tropicana Detention Basin, the flow data from the junctions (junction conveys the flow in the model from upstream to downstream) at these locations were used. At the outlet of the watershed (Flamingo Wash at Nellis Boulevard),
the observed flow was also compared. The model results were mixed. However, the
model does an excellent job on overall timing of the peak.

Figure 4-28 compares the hydrographs at Upper Flamingo Detention Basin. The
model generated two distinct peaks. The timing of the first peak for both of the radar data
was at 1:30 A.M. on December 29th, 2004. Whereas, it was observed at 2:00 A.M. for
gage rainfall data. On the other hand, the second peak occurred exactly at the same time
(5:00 A.M.) with all the three types of rainfalls. The comparison of hydrographs further
revealed that the modeled volume with radar rainfall is much higher than the gage
rainfall. Among the hydrographs of radar rainfall, the 1 KM resolution radar data appear
to generate more flow volume especially during the second peak. In both the peaks, the
recession limbs of the hydrographs appear to show a close match among each other as
compared to the rising limbs.

![Hydrographs comparison](image)

Figure 4-28. Comparison of the model flows using radar and gage precipitation data at
Upper Flamingo Detention Basin.
The next point of interest was Tropicana Detention basin. Figure 4-29 shows the hydrographs for 1 KM, 2 KM, and gage rainfall data. The two peaks were distinct at this location as well. The first peak with both radar data was recorded at 1:45 A. M. on December 29th, 2004. With gage precipitation data, the peak occurred at 2:00 A. M. The second peak occurred at 5:00 A. M. for all the three rainfalls. As with the above case, at this location also the 1 KM radar data produced more flow in the second peak and the recession limb of all the three hydrographs showed close match as compared to rising limb.

![Graph showing hydrographs for different rainfalls at Tropicana Detention Basin](image)

Figure 4-29. Comparison of the model flows using radar and gage precipitation data at Tropicana Detention Basin.

The model flow was also compared at the outlet of watershed (Flamingo Wash at Nellis Boulevard). Figure 4-30 shows the model flow for radar and gage precipitation data along...
with the observed flow recorded by the USGS gage # 094196781. The modeled flow volume appears to be higher than the observed flow volume. 1 KM radar data produced the highest flow volume, followed by slightly lower volume with 2 KM radar data. Gage rainfall data produced much less volume. However, in overall the time to peak was observed at the same time with all the rainfalls, which also matched very well with the observed peak. Like the above two cases, the model produced two peaks at the watershed outlet as well. The first peak for observed flow was recorded at 2:15 A.M. on December 29th, 2004. The radar data also peaked at the same time whereas; the gage data produced peak at 3:15 A.M. For the second peak, the time to peak was observed at 5:45 A.M. for the radar and gage rainfall, whereas observed peak flow was recorded at 5:30 A.M. The slope of the rising and recession limbs of all the hydrographs also matches well among each other.

Figure 4-30. Comparison of the model flow using radar and gage precipitation data as well as observed flow at Flamingo Wash at Nellis Boulevard.
4.4.3 Summary of Radar Rainfall Modeling

The model result using the radar rainfall data does an excellent job of predicting the time to peak. The model flow of radar rainfall was compared with the model flow of gage rainfall at three locations in the watershed. Since the observed flow was only available at the outlet of watershed, it was also considered for comparison. The model produced two distinct peaks. These peaks occurred initially at the upstream location and followed successively at the downstream locations. Most of the time the time to peak of radar data was found to match with that of the gage data. At the outlet of watershed, the time to peak for radar data matched very well with the observed flow.

Although the model successfully predicted the time to peak, it could not predict the peak flow well. The model overestimated the peak flow and it was even higher than the gage rainfall data. This is due to the fact that the radar data produced higher rainfall depth in the watershed (Figure 4-31) and unlike the gage rainfall the radar rainfall was recorded by each individual grid and was associated with fewer uncertainties than the gage data since it was derived for each subbasin while being synchronized with the gage rainfall data.
Figure 4-31. Total rainfall depth of the radar and gage rainfall data.
CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The main objective of this research was to develop a real-time flood forecasting model. The research was conducted with respect to the three questions that were posed in section 1.3. This chapter provides the answers to these questions followed by conclusions and finally recommendations for future work.

The hydrologic model in this research was developed using the existing HEC-1 MPU model. The basin model of the HEC-1 model represented the ultimate build-out condition in the watershed and the meteorologic model consisted of a design storm for a 100-year return period. The model was basically meant for simulating the peak flows for the purpose of designing flood control structures. Since it contained the design storm, the HEC-1 model was originally inappropriate for handling the real-time rainfall data. Therefore, this research developed a new meteorological model.

The research first simulated the real-time gage rainfall data. Three rainfall events of 28-29 December 2004, 23-24 November 2004, and 24 July 2005 were considered. The model results were compared with the observed data at various locations in the watershed. The results of these rainfall events were mixed. The model overestimated the peak flow for all the storm events at Upper Flamingo Detention basin and Tropicana Detention Basin. At the watershed outlet, the model overestimated the peak flow for December 2004 storm events only and underestimated the peak flow for November 2004 and July 2005 storm events. In other words, the model overestimated 7 out of 9 comparisons (based on comparison of three storm events at three places). The
underestimation of peak flow was only at the watershed outlet (with USGS observed flow). This underestimation can be attributed to several factors. The USGS gage may not be accurately functioning at the time of the rainfall and the supplied flow data may be the estimated unit discharges. Similarly, the stream and the outlet gage may have been clogged with debris during the storm events. There could be several other reasons that might have associated uncertainty with the observed flow data at the watershed outlet which prevented the model from overestimating the observed flow.

Though the observed flow was not 100% overestimated, the time to peak at various places in the watershed was accurately simulated for all the three rainfall events. It was observed that the model peak flow was proportional with the total rainfall depth produced by the respective storms. The December 2004 storm produced the largest rainfall depth followed by November 2004, and July 2005 storm. Hence, the peak flow produced by December 2004 rainfall was higher, followed by November 2004, and then by July 2005 storm. Therefore, the preliminary results were encouraging. Though the shape of the model hydrograph matched only adequately with the observed hydrograph, the time to peak matched very well indicating that the existing HEC-1 MPU model can be used to predict the time to peak.

The real time rainfall data used in the research was obtained from the FTRS of CCRFCD. A visual analysis of the rainfall records showed that the rainfalls are recorded for every 0.04 inch of rainfall depth. In other words they are recorded on the basis of increase in rainfall depth, not on an even time increment. However, HEC-HMS requires the rainfall in equal time intervals. Distributing the rainfall in equal time intervals was a major issue in this research. This was accomplished using the VLOOKUP function in MS
Excel. The function does not interpolate the rainfall value but it looks for the nearest rainfall value based on the interval of time period. Since the distribution of rainfall was an approximation, it contained a major source of uncertainty. This was evident from the fact that the gage data could not predict the smaller peaks that occurred before, after and in-between the two major peaks (see Figure 4-29). This proved that the real-time gage rainfall data could be used for predicting the time to peak.

As described above, the model underestimated the peak flow for November 2004 and July 2005 storm event. These events could not be used for further analysis. Therefore, the December 2004 rainfall was considered for further analysis. Sensitivity analysis was performed with this storm to get the value of initial loss in order to develop the model resembling the existing basin condition. An initial loss value of 0.5 inch was the optimum value that was obtained without affecting the time to peak and distorting the overall shape of the hydrograph. This value was included in the basin and the modeling was done using the radar rainfall data having a resolution of 1 KM and 2 KM.

The radar rainfall data was derived for individual basin in the watershed. The meteorologic model developed for gage rainfall data was not applicable with radar data. Hence, a new meteorologic model was created to simulate the radar rainfall data. The model hydrographs for 1 KM and 2 KM resolution radar rainfall was compared with the gage rainfall data and observed data at various locations. The radar rainfall produced slightly improved results over the gage rainfall results. The comparison for time to peak with radar data was close to that of the gage rainfall data. In other words the time to peak was more accurately predicted with radar data. However, the model produced more peak rainfall amount. This difference in peak rainfall between the gage data and radar data can
also be attributed to some degree to the different meteorologic model used with these two
different sets of data. More importantly this could be due to the nature of the gridded
rainfall data, as each grid records the rainfall in the watershed. The simulation of 1 KM
and 2 KM radar data, however, did not reveal significant difference in model result. Both
the rainfall data predicted the time to peak with equal precision. The peak flow produced
by 1 KM radar data was also only slightly more than the 2 KM data.

5.1 Conclusion

This research systematically handled various steps including data acquisition and
processing, sensitivity analysis, and model development using different approaches.
Although the basin model represented ultimate build-out condition in the watershed and
the basin was still experiencing considerable human interference due to population
growth during the study period, the model results, especially the ability of the model to
predict the time to peak at various places in the watershed with different rainfall data was
were well represented. The model was found to forecast the time to peak very well but
not the peak rainfall.

The model results were therefore encouraging and it was seen that the existing model
could be used to develop the real-time flood forecasting model.

5.2 Recommendation for Future Study

The results of the research and conclusions drawn from these results indicate that
additional work must be accomplished to develop an efficient real-time flood forecasting
model. Some of these are described below.
The use of gage rainfall data was a major challenge for efficient modeling. The methodology used in this research to process the gage rainfall data was tedious and time consuming. The methodology was well suited to test the existing HEC-1 model's capability to flood forecasting. It may not provide enough lead time for emergency response during the time of intense rain event. Moreover, the processing was also not free from uncertainties. Therefore the efforts should be directed to develop an interface data model or software capability that could more efficiently process the real time rainfall data for immediate error free use of rainfall data in the model.

Though the model represented ultimate build-out condition in the watershed, it predicted the time to peak very well. However, it could not predict the peak flow. The incorporation of the result of sensitivity analysis into the model was also not fruitful in predicting the peak flow. The future work should focus on addressing this shortcoming of the model. Since the southwest portion of the Las Vegas Valley was still undeveloped during the study period, the model may better predict the peak flow if the basin model for this area were changed to better represent the existing condition.

Additional work needs to be implemented to integrate this hydrologic model with the hydraulic model and develop a decision support system. The author believes that carrying out these major recommendations would help to development a more efficient hydrologic model for flood forecasting. The authors further hopes that this thesis document disseminates the required information to extend modeling in other watersheds of the Las Vegas Valley to develop an integrated hydrologic model for real-time flood forecasting in the desired location.
APPENDIX 1

STATION ID AND STATION NAME OF THE RAIN GAGES USED IN THE FLOOD THREAT RECOGNITION SYSTEM
<table>
<thead>
<tr>
<th>Station ID</th>
<th>Name of the Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>4084</td>
<td>Las Vegas Wash at Sahara Avenue</td>
</tr>
<tr>
<td>4274</td>
<td>Downtown Las Vegas</td>
</tr>
<tr>
<td>4314</td>
<td>Blue Diamond Ridge North</td>
</tr>
<tr>
<td>4324</td>
<td>Red Rock Canyon</td>
</tr>
<tr>
<td>4334</td>
<td>Upper Flamingo Wash 1</td>
</tr>
<tr>
<td>4364</td>
<td>Flamingo Wash at Torrey Pines</td>
</tr>
<tr>
<td>4374</td>
<td>Flamingo Wash at Eastern</td>
</tr>
<tr>
<td>4454</td>
<td>Warm Springs at Jones</td>
</tr>
<tr>
<td>4304</td>
<td>Blue Diamond Ridge South</td>
</tr>
<tr>
<td>4369</td>
<td>Flamingo Wash at Decatur</td>
</tr>
<tr>
<td>4394</td>
<td>Flamingo Wash at Nellis Blvd</td>
</tr>
<tr>
<td>4349</td>
<td>Upper Flamingo Detention Basin</td>
</tr>
<tr>
<td>4379</td>
<td>Van Buskirk Detention Basin</td>
</tr>
<tr>
<td>4384</td>
<td>Desert Inn Super Arterial</td>
</tr>
<tr>
<td>4474</td>
<td>Tropicana Detention Basin</td>
</tr>
<tr>
<td>4329</td>
<td>Brownstone Canyon</td>
</tr>
<tr>
<td>4434</td>
<td>Beltway Channel at Buffalo</td>
</tr>
<tr>
<td>4359</td>
<td>Lakes Detention Basin</td>
</tr>
<tr>
<td>4319</td>
<td>Beltway Channel at Town Center</td>
</tr>
<tr>
<td>4309</td>
<td>Desert Inn Detention Basin</td>
</tr>
<tr>
<td>4354</td>
<td>The Lakes</td>
</tr>
<tr>
<td>4339</td>
<td>Beltway Channel at Peace Way</td>
</tr>
<tr>
<td>4414</td>
<td>Blue Diamond Detention Basin</td>
</tr>
<tr>
<td>4399</td>
<td>Flamingo Wash Near Mojave</td>
</tr>
<tr>
<td>4484</td>
<td>Tropicana Wash at Swenson Avenue</td>
</tr>
<tr>
<td>4344</td>
<td>Red Rock Detention Basin</td>
</tr>
<tr>
<td>4424</td>
<td>F-1 Channel</td>
</tr>
<tr>
<td>4444</td>
<td>R4 Detention Basin</td>
</tr>
<tr>
<td>4449</td>
<td>R4 Channel</td>
</tr>
<tr>
<td>4574</td>
<td>Flamingo Wash Near Spencer</td>
</tr>
<tr>
<td>4409</td>
<td>F-2 Debris Basin</td>
</tr>
<tr>
<td>4404</td>
<td>F-1 Debris Basin</td>
</tr>
</tbody>
</table>
APPENDIX 2

RATING CURVES FOR UPPER FLAMINGO DETENTIONS BASIN, TROPICANA DETENTION BASIN, AND FLAMINGO WASH AT NELLIS BOULEVARD
1. Rating curve for the Upper Flamingo Detention Basin:

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Q(cfs)</th>
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<tr>
<td>2.5</td>
<td>12</td>
</tr>
<tr>
<td>3.5</td>
<td>41</td>
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<tr>
<td>4.5</td>
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<td>11.5</td>
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<tr>
<td>25.5</td>
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<td>20100</td>
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<td>30.5</td>
<td>46200</td>
</tr>
<tr>
<td>31.5</td>
<td>55600</td>
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</table>

2. Rating curve for the Flamingo Wash at Nellis Blvd.:

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Q(cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.1</td>
<td>15</td>
</tr>
<tr>
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<td>67</td>
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<tr>
<td>0.60</td>
<td>200</td>
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<td>1000</td>
</tr>
<tr>
<td>2.2</td>
<td>1500</td>
</tr>
<tr>
<td>3.0</td>
<td>2000</td>
</tr>
<tr>
<td>3.4</td>
<td>2500</td>
</tr>
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<td>3.9</td>
<td>3000</td>
</tr>
<tr>
<td>4.0</td>
<td>3200</td>
</tr>
<tr>
<td>5.1</td>
<td>4500</td>
</tr>
<tr>
<td>6.2</td>
<td>6000</td>
</tr>
</tbody>
</table>

3. Rating curve for the Tropicana Detention Basin:

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Q(cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>3.39</td>
<td>34</td>
</tr>
<tr>
<td>4.31</td>
<td>161</td>
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<td>8.41</td>
<td>203</td>
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<td>12.18</td>
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</tr>
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<td>13.40</td>
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</tr>
<tr>
<td>18.74</td>
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<tr>
<td>25.31</td>
<td>400</td>
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<tr>
<td>38.43</td>
<td>500</td>
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</table>
APPENDIX 3

COMPARISON OF THE ACTUAL RAINFALL FROM THE FTRS RAIN GAGES
AND ESTIMATED RAINFALL FOR DECEMBER 28-29 STORM IN THE
FLAMINGO TROPICANA WATERSHED
Figure. Actual and Estimated Rainfall for Gage 4084

Figure. Actual and Estimated Rainfall for Gage 4274
Figure. Actual and Estimated Rainfall for Gage 4314

Figure. Actual and Estimated Rainfall for Gage 4324
Figure. Actual and Estimated Rainfall for Gage 4334

Figure. Actual and Estimated Rainfall for Gage 4364
Figure. Actual and Estimated Rainfall for Gage 4374

Figure. Actual and Estimated Rainfall for Gage 4454
Figure. Actual and Estimated Rainfall for Gage 4304

Figure. Actual and Estimated Rainfall for Gage 4369
Figure. Actual and Estimated Rainfall for Gage 4394

Figure. Actual and Estimated Rainfall for Gage 4349
Figure. Actual and Estimated Rainfall for Gage 4379

Figure. Actual and Estimated Rainfall for Gage 4384

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Figure. Actual and Estimated Rainfall for Gage 4474

Figure. Actual and Estimated Rainfall for Gage 4329
Figure. Actual and Estimated Rainfall for Gage 4434

Figure. Actual and Estimated Rainfall for Gage 4359
Figure. Actual and Estimated Rainfall for Gage 4319

Figure. Actual and Estimated Rainfall for Gage 4309
Figure. Actual and Estimated Rainfall for Gage 4354

Figure. Actual and Estimated Rainfall for Gage 4339
Figure. Actual and Estimated Rainfall for Gage 4414

Figure. Actual and Estimated Rainfall for Gage 4399
Figure. Actual and Estimated Rainfall for Gage 4484

Figure. Actual and Estimated Rainfall for Gage 4344
Figure. Actual and Estimated Rainfall for Gage 4424

Figure. Actual and Estimated Rainfall for Gage 4444

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Figure. Actual and Estimated Rainfall for Gage 4449

Figure. Actual and Estimated Rainfall for Gage 4574
Figure. Actual and Estimated Rainfall for Gage 4409

Figure. Actual and Estimated Rainfall for Gage 4404
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WRC Engineering Inc. (1989). Collection and Analysis of Historical Rainfall Data and Patterns to Support Continued Use of Changes to the Adopted Clark County Regional Flood Control District’s Design Rainfall For Drainage Facility Design and Analysis.


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