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INFORMATION TECHNOLOGIES IN WATER RESOURCES:
MODELING FLOOD CONTROL ALTERNATIVES FOR THE CLEAR CREEK
WATERSHED WITHIN A GIS FRAMEWORK

by

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A THESIS SUBMITTED
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ABSTRACT

Information Technologies in Water Resources: Modeling Flood Control Alternatives for the Clear Creek Watershed within a GIS Framework

by

Jude A. Benavides

Recent technological advancements in hydrologic science and geographic information systems (GIS) have permitted the rapid and accurate analysis of flood control options in the Clear Creek watershed. Models were developed using the Hydrologic Engineering Center’s latest hydrologic and hydraulic programs, including HEC-HMS, HEC-RAS and HEC-GeoRas. These models were used to develop alternatives to large-scale channelization, including limited channelization, floodplain property buyouts, and a combination approach. NEXRAD radar was used to compliment an inadequate rain gage network. The above tools were coupled with available floodplain data and a GIS program (ArcView) to produce floodplain mapping for each scenario.

The effects of six different limited channelization schemes were explored. Cost estimates were calculated for properties within various return frequency floodplains. Results show that the combination of one of the investigated channelization schemes with buyouts of properties in the residual floodplain was the most viable from the perspectives of flood control and economics.
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Chapter 1. Introduction

Flooding and flood damages have continued to increase at alarming rates throughout the country in recent years. This has occurred despite repeated engineering efforts to halt this most expensive of natural disasters in the United States (NWF, 1998). These efforts are coordinated by a variety of different government agencies, ranging from local county flood control districts to the largest federal agency responsible for flood control – the United States Army Corps of Engineers (Corps). In many instances, flood control plans are stalled at various levels of implementation due to local dissatisfaction with one or more elements of a federally sponsored plan.

The Clear Creek watershed, located south of Houston, Texas, is currently experiencing increased flooding problems due to the lack of implementation of a specific, watershed-wide flood control plan. A nearly 35-year old, congressionally approved channelization scheme has finally been recalled by the Corps for additional review. Strong opposition by both local government agencies and concerned citizens groups over the detrimental environmental impacts of the plan resulted in the Corps agreeing to conduct a General Re-evaluation Report, or GRR, that would essentially send the plan to the preliminary stages of development and design for restudy (HCFCD, 1997; USACE, 2000a).

This three-year long review period created an opportunity for research into the use of recently developed technologies. Some of these technologies include the latest Hydrologic Engineering Center’s (HEC) hydrologic and hydraulic programs, Next
Generation Radar (NEXRAD), and the continued inclusion of Geographic Information Systems (GIS) in flood control modeling.

A GIS based approach to flood control is now possible due to the ever-increasing amount of more accurate and higher-resolution digital hydrologic data. An analysis of several different flood control options ranging from the traditional channelization approach to the less frequently employed non-structural alternatives would become less costly and time consuming should GIS and the other above mentioned technologies be incorporated in any overall flood control plan.

1.1. Significance of Research

The new modeling programs released by the HEC are slowly replacing their predecessors in the field. HEC's recently released Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS), along with their geospatial counterparts, HEC-GeoRas and HEC-GeoHMS are designed to work with a wide variety of recently available digital data (USACE, 2000h). However, their acceptance by the engineering community has been slow and any additional research proving the capabilities of these programs would go far toward increasing their application (Flores, 2000).

Additionally, GIS continues to play a more prominent role in watershed analysis ranging from flood control to drought management. Previous barriers to GIS implementation have included the lack of available digital data, the overall financial expense associated with GIS modeling, and the lack of cross-discipline cooperation between the engineering and GIS communities (Maidment and Djokic, 2000). These barriers are quickly being dissolved. Continued advances in personal computers have
permitted once extremely expensive GIS studies to be conducted at the desktop level (Foresman, 1998). This has spurred the transfer and sharing of digital databases between engineering firms, governmental agencies and research institutions at unprecedented levels. And this, in turn, has forced new avenues of communication between the GIS and more traditional engineering fields (Maidment and Djokic, 2000).

1.2. Overview

The combination of GIS technology with the previously mentioned programs should prove to be a powerful modeling approach for the Clear Creek watershed and other areas. This approach was accomplished by following the general methodology shown in Figure 1.1. The figure illustrates how the variety of tools and modeling programs utilized in this research were combined to transform a given rainfall input into a

![Diagram](image)

Figure 1.1: Schematic of Overall Methodology
corresponding digital floodplain map for specific portions of the Clear Creek watershed.

Any hydrologic and/or hydraulic analysis must begin with either an actual or theoretical rainfall input (step 1). This rainfall input is converted to runoff via a hydrologic model (HEC-HMS), resulting in a corresponding hydrograph of flow versus time (step 2). The peak flows of each of these hydrographs are used as input to a hydraulic model (HEC-RAS) to calculate the resulting water surface elevations along particular reaches (step 3). The HEC-GeoRas program then allows for the conversion of these elevations to a unique inundation area known as a floodplain (step 4). Finally, additional GIS data can be combined with the resulting digital data displays to analyze a wide variety of flood control alternatives in a quick and efficient manner (step 5).

It is important to mention that while a full watershed-wide hydrologic analysis was completed in this study, the hydraulic analysis portion focused only on a particular area of interest. The criteria for selecting this area of interest are discussed later in this work. Narrowing the focus of the hydraulic analysis was done to reduce the financial expenditure associated with data gathering. However, the overall objective of applying these new technologies within a GIS framework is still met. Additionally, the overall approach described in this research may be readily applied to other areas throughout the watershed after the required digital data is obtained.

1.3. Research Objectives

The following is a list of the research objectives to be pursued in the course of this study:
1. Evaluate the overall flooding problem in the Clear Creek watershed and determine a suitable area of interest within the watershed for detailed study.

2. Utilize NEXRAD rainfall estimates for rainfall input into a hydrologic model of the Clear Creek watershed.

3. Develop and illustrate the utility of a hydrologic model for the Clear Creek watershed in the improved Hydrologic Modeling System (HMS) format.

4. Develop and illustrate the utility of a hydraulic model in the River Analysis System (RAS) format for determining water surface elevations in the study area.

5. Utilize available GIS data, ArcView and the HEC-GeoRAS software to develop digital floodplains that can be used as both qualitative and quantitative measures of the effectiveness of various flood control options.

6. Combine the above models to investigate the effectiveness of two proposed flood control alternatives including limited channelization and floodplain property buyouts.
Chapter 2. Study Area

The following section introduces the study area referred to as the Clear Creek watershed. Several relevant environmental and hydrologic characteristics of the watershed are highlighted. A brief historical overview of the flooding problems that have traditionally plagued the region is also presented.

2.1. Location

The Clear Creek watershed is a 260 square mile area located in southeast Texas, adjacent to the southern border of the City of Houston. Clear Creek is a tidally influenced bayou that meanders through four counties (see Figure 2.1) and several municipalities before terminating as it enters Clear Lake. Clear Lake also receives flows

Figure 2.1: Location of the Clear Creek Watershed
from the Armand and Taylor bayous, which together with Clear Creek comprise the Clear Creek watershed area. However, this research focuses only on the 164 square miles of the watershed drained directly by Clear Creek and its tributaries. The major tributaries draining directly to Clear Creek are: Chigger Creek, Cowart Creek, Turkey Creek, Marys Creek and the Hickory Slough. (See Figure 2.2)

2.1.1. Watershed Characteristics

The Clear Creek waterway is subject to extensive flooding as a result of mild slopes, intense rainfalls, and predominantly clay soils (Vieux and Bedient, 1998). The watershed is primarily composed of relatively flat coastal plains with elevations varying around sea level near Clear Lake to about 75 feet msl at its western end, resulting in average slopes of 0.1% to 1%. Clear Creek experiences approximately 47 inches of
annual rainfall, with the vast majority of this rainfall being associated with tropical activity. This results in relatively short-duration, high-intensity storms that generate large amounts of direct runoff with extremely short times to peak. Soil composition consists mostly of clayey, class D hydrologic soils with extremely low infiltration rates on the order of 0.05 in/hr (USACE, 1982).

This mostly natural channel originates in Fort Bend County and flows generally eastward for approximately 47 river miles, forming the boundary between Harris-Brazoria and Harris-Galveston Counties. The channel currently varies from a narrow, nearly dry channel at its upper limits to several hundred feet wide at its lower end (see Figure 2.3). In general, all streams in the watershed are meandering and have small channel capacities relative to the area drained. Consequently, overbank conditions are not uncommon throughout the watershed and the 100-year floodplain is naturally extensive, especially in the
upstream regions of the watershed (USACE, 1982; HCFCD, 1997).

2.1.2. History of the Flooding Problem

Major flood events (equal to or in excess of a 25-year storm) throughout the 20th century have made flooding a primary area of concern in the watershed, with major events occurring in the following years: 1932, 1940, 1942, 1946, 1959, 1973, twice in 1979, and most recently in 1994 (HCFCD, 1997). Watershed wide floods have resulted primarily from tropical storms and hurricanes. In July of 1979, Tropical Storm "Claudette" caused an unprecedented $95 million worth of damages throughout the watershed by flooding over 5,000 structures (USACE, 1982). This enormous storm was in excess of a 100-year event throughout areas in and around the Clear Creek watershed. In fact, this event still claims the national 24-hour rainfall record of 42 inches, occurring near Alvin, Texas, approximately 7 miles south of the Clear Creek watershed.

A request for federal flood control assistance has been in place since 1964, when statements were submitted by state and local government officials as well as concerned private citizens relating to the severe flooding along Clear Creek. To date, local flood control actions on the county scale have attempted to address the flood issue with some positive results. However, the Corps has failed to create and fully implement a federal flood control project acceptable to all local stakeholders (HCFCD, 1997; Blackburn, 2000). It is this difficulty in analyzing and determining various flood control options, complicated by the several factors presented in this section, which inspired this research.

2.2. Current Flood Control Solutions
Since the inception of a federal flood control project for the Clear Creek area, the Corps had until only recently, desired that the channelization or "big ditch" approach be the principal flood control option for this watershed. Local sponsors as well as the Corps have made various changes to the original 100-year channel design originally conceived in the late 1970's, with some changes relying more heavily on channelization than others. Until recently, however, all proposals that have been given serious consideration by the Corps have remained channelization schemes at heart (HCFCD, 1997; Blackburn, 2000). This section provides a discussion of the original and modified Corps plans in addition to the latest local sponsor plan. Advantages and disadvantages of each of these proposals are discussed in an attempt to highlight what plan elements should be maintained in any preferred solution to be researched.

2.2.1. U.S. Army Corps of Engineers' Proposal

The Clear Creek Federal Flood Control project has had a lengthy and involved history of formulation and reformulation. The original plan, with funding initially approved by Congress in 1968, consisted of an earthen, grass-lined channel that would have extended the full length of the creek from the upper end of Clear Lake, westward to its origin in Fort Bend County. The result would have replaced approximately 41 miles of existing, winding channel with a 31-mile relatively straight channel designed to accommodate 100-year flows. The plan remained dormant until 1979 when Tropical Storm Claudette emphasized the need for action. Despite this impressive event, post authorization planning studies conducted by the Corps found that the project lacked broad public support because of the negative aesthetic and environmental impacts associated
with the original channel design (USACE, 1982). As a result, the channelization plan was reduced in scope and was finally initiated in 1986 when the Galveston and Harris County Flood Control Districts signed in support of the plan as local sponsors. Local sponsor agreement was required for any federal flood control actions by the then recently passed Water Resources Development Act of 1986. The Brazoria County Drainage District refused to sign as a local sponsor and channel improvements no longer extended to Fort Bend county, thus effectively and immediately halving local support for the plan.

The reduced-scope project, now designed to only accommodate a 10-year event, affected a smaller stretch of the creek (shown in Figure 2.2) but still experienced similar difficulties in implementation as its predecessor. The only portion of the project successfully completed was the construction of a second outlet (in addition to the naturally existing connection) from Clear Lake to Galveston Bay, designed to prevent flooding along the lake which was expected to be caused by the additional flow generated by the improved channel. The remainder of the plan again encountered stiff opposition by local citizens groups, communities and government agencies. The concerns shared by these sponsors are summarized below (HCFCD, 1997):

- The current federal project is based on outdated information and fails to utilize recently available technologies and should not be continued without further review.

- An enlarged Clear Creek channel along with increased upstream discharges associated with future development could overpower the second outlet resulting in increased flood levels in the Clear Lake area. Additional concerns included the possibility of a high tide or storm surge associated with a tropical
storm event rendering the second outlet useless and again resulting in flood levels around the Clear Lake area higher than those predicted by current models.

- The current (10-year channel) federal project still has an unnecessarily and unacceptably harsh impact on the environment.
- Channelization of the natural stream could be reduced or avoided if more project elements were employed, especially regional detention, natural corridor bypasses, reduced channel rectification and floodplain property buyouts of selected structures. Extensive research into the results of the combination of any number of these alternatives would also be required.

2.2.2. Harris County and Galveston County Flood Control Districts' Proposal

In an attempt to address the above concerns, the Harris County Flood Control District (HCFCD) initiated a six-month project review that would be conducted in conjunction with the Corps and the Galveston County Flood Control District (HCFCD, 1997). The review was to determine the soundness of the current project with respect to flood control effectiveness and environmental impact as well as to provide an opportunity for the public to express its concerns and desires. The review would then choose from one of the three following courses of action. First, was to continue the current Corps project as planned. Second, was to completely abandon the project in favor of another approach, resulting in the loss of $75 million in federal funding support that was approved specifically for the original plan. The last was to significantly change the Corps channelization plan by implementing a variety of additional flood control methods.
but ensure that the final agreed upon course of action would remain "within reasonable similarity" to the original Corps plan in order to retain federal funding (HCFCF, 1997).

The outcome of the review, known as the HCFCF Plan, included a limited number of property buyouts, in-line detention ponds and the use of by-pass channels but still allowed channelization along significant portions of Clear Creek. HCFCF dismissed, due to excessive cost, alternative options known as "pure alternatives" which relied solely on one particular type of flood control. This excessive cost was a result of including the possible loss of federal funding and other "externalities" as a cost associated with that alternative.

The dismissal of the "property buy-out" only option is a perfect example of this type of financial comparison. The cost of the current COE channelization project was estimated at $129 million, with the cost-sharing agreement being $75 million of this amount provided by the federal government and the rest by local sponsors. During the re-evaluation, the buyout-only option was estimated to cost $60 million to purchase 400 homes in the 100-year floodplain near Friendswood. However, the "full" cost of this alternative, as defined by the review was considered to be the $60 million base amount plus an additional cost of $40 million associated with the second outlet and other improvements already completed as part of the original plan. The resulting $100 million price tag, combined with the potential loss of federal funding, made this singular option undesirable (HCFCF, 1997). As an additional concern, plans rejected due to cost were not studied or investigated any further. Thus, any other potential benefits from these plans, such as preservation and reduced environmental impacts, were not studied in detail.
As a result of the above, many opponents of the HCFCD plan argued that the new project was a veiled floodplain reclamation scheme seeking to allow further development of upstream floodplains instead of a well-founded flood control alternative. (Blackburn, 2000). Additionally, this latest plan still lacked watershed-wide support. Continued dissatisfaction with all of the previously discussed flood control plans by local sponsors forced the Corps to initiate a General Re-evaluation Report that would send the original channelization plan to the preliminary stages of development and design for restudy (HCFCD, 1997; USACE, 2000a).
Chapter 3. Background and Literature Review

The Clear Creek flooding problems discussed in the previous chapter are shared by many different geographic locations throughout the United States. Since many of these problems are rooted in the federal policy established nearly a century ago, it is important to discuss some background information at the national level.

The beginning sections of this chapter provide the reader with some essential background information concerning flood control policy and the agency responsible for enforcing these policies, the United States Army Corps of Engineers (Corps). The chapter continues by discussing several of the options available for flood control often used by the Corps and other agencies. The final sections discuss specific literature reviews of Next Generation Radar (NEXRAD) and Geographic Information Systems (GIS) applications in the hydrologic and hydraulic fields.

3.1. New Direction for the U.S. Army Corps of Engineers

The year 2000 brings with it an interesting twist to the future of the United States Army Corps of Engineers (Corps) as it considers major changes in its planning and operational processes to more fully integrate environmental protection, local sponsor requests and quicker action in answering the nation's water-related problems. Many of the Corps' previous attempts at solving issues such as flood damage reduction have come under heavy scrutiny for their ineffectiveness, cost, time-to-completion, and environmental impacts. Concerns are not focused solely on this portion of the Corps' responsibilities but extend to other types of projects as well. These include navigation,
shoreline/beach protection, and water supply projects (Reisner, 1993). Sources of dissatisfaction range from local interest groups and stakeholders to national organizations, both public and governmental.

In response to these concerns, the Corps asked The National Research Council’s Water Science and Technology Board to conduct a study of its planning and operational processes. Although not specifically tasked with evaluating the above concerns with Corps projects, the Board’s study can be used as an excellent indicator and confirmation of the need for change within this organization.

After reviewing this report and others, it is clear that the Corps is not only in need of change but is showing significant promise toward that end goal. This section will begin with a brief discussion of the historical factors influencing the current planning scheme within the Corps. A discussion of the current processes follows and introduces some of the major operational limitations and regulatory restrictions that have constrained previous and current Corps projects.

3.1.1. The Evolution of Corps Programs and Federal Water Policies

The Corps has been influential in water resource development since the early-1800’s. An 1824 Supreme Court decision (Gibbons v. Ogden) gave the Corps generous authority and broad responsibility over the nation’s waterways when Chief Justice John Marshall declared that the “federal power to regulate interstate commerce carried with it a similar federal authority over navigation” (Rogers, 1993). In 1850, the U.S. congress directed the Corps to “determine the most practical plan” to control flooding along the lower Mississippi River. Two strongly differing reports emerged from the study. The
first, by Captain Andrew Humphreys, promoted a levees-only strategy along the lower river to the exclusion of other options. The second report, by a respected civil engineer by the name of Charles Ellet, was a significantly more diversified and comprehensive strategy that included the creation of upstream storage reservoirs and enlarged river outlets in addition to the strengthening of levees in critical areas along the river. Eleven years later, in 1861, the Corps opted for the levees-only strategy and set the stage for nearly a century and a half of heated debate over both the role of the federal government in solving regional water resource problems and the best alternatives to employ toward their solution.

The Corps has continued to play an important role in forming a vast majority of the water resource systems and policies in the United States. The Corps predates all other federal water planning agencies, such as the U.S. Bureau of Reclamation, and is the only agency to have sponsored projects in all 50 states. Therefore, any change in its programs and planning procedures will most definitely have sweeping impacts throughout the nation.

Important legislation was passed throughout the late 1800s up until the 1960s that defined the scope, power and authority of the Corps. Among these were the 1899 River and Harbor Act, which gave the Corps authorization to monitor and prohibit the dumping of dredged material in the nation’s navigable waterways. The Federal Water Power Act of 1920 and the River and Harbor Act of 1925 established hydroelectric power projects under the Corps’ auspices in addition to improvements in “navigation, flood control and irrigation on the navigable streams of the U.S. and their territories” (U.S. Congress, 1925). The Corps’ benefit-cost procedures were developed in the 1930s under the
authority of Section I of the 1936 Flood Control Act. This act specified, "the federal
government should improve or participate in the improvement of navigable waters or
their tributaries, including watersheds thereof, for flood control purposes if the benefits to
whomsoever they may accrue are in excess of the estimated costs, and if the lives and
social security of people are otherwise adversely affected" (U.S. Congress, 1936). This
act would subject all future Corps projects to this deceptively simple test. Over the next
60 years, the Corps' benefit-cost analysis has been refined on a near continual basis but is
still the focus of much debate.

Several other acts, committee documents and circulars up until the late 1960s,
provided an avenue for ambitious water resources projects that, although still subject to
an ever changing form of the benefit-cost test, began to change the focus of the Corps and
the complexity of its mission. It can be discerned from Figure 3.1 that 1969 was a critical

Figure 3.1 : The Increasing Mission Complexity and Changing Water Resource
Mission of the United States Army Corps of Engineers
turning point with respect to the way the Corps viewed environmental protection as its water resource mission transformed from a mostly control and allocating strategy to one more sensitive to environmental protection and eventually, environmental restoration (NRC, 1999a).

The Water Resources Development Act (WRDA) of 1986 greatly changed the way new projects within the Corps would be studied and conducted (NRC, 1999b). The act established a framework that promoted federal-nonfederal partnerships and local sponsors were given a greater role in project planning. Subsequent WRDA's further encouraged local sponsorship and participation. As their participation level rose, so did their cost-sharing percentage, now ranging from 25% to 50% of the total project costs. As their cost-sharing percentage rose, local stakeholders began to feel that their concerns should play a more important role in any flood control plan (NRC, 1999a).

Local sponsors have recently been able to force a fundamental shift in project emphasis; calling, essentially, for a coequal objective system that stresses both environmental quality and economic development at all levels when possible. This view is strongly supported by Brigadier General Gerald E. Galloway, U.S. Army, Executive Director of the Interagency Floodplain Management Review Committee (USEOP, 1994). One of the best methods to ensure that environmental concerns are adequately addressed in any flood control plan is to actively pursue the inclusion of nonstructural methods of flood control whenever possible (USEOP, 1994).

3.2. The Flood Control Approach: Structural, Nonstructural or Both
The three basic approaches to alleviation of flood damages are: first, to attempt to control floodwaters with the construction of flow-channeling levees, floodwalls, bypasses, or other channel modifications and conveyance works; second, to adjust the use of flood-prone lands to the existing flood hazard by a variety of measures; and last, to control and adjust by combinations of these protective structural works and non-structural measures (NRC, 1995).

People have defined flooding in many different ways, often times in such a manner to trivialize the complex factors contributing to a flood condition. Chow defines flooding in a fairly limited and restrictive manner: “A flood is a relatively high flow which overtaxes the natural channel provided for the runoff” (Chow et al., 1988). A more complete definition is as follows: “A general and temporary condition of partial or complete inundation of normally dry land areas from the overflow of river and/or tidal waters and/or the unusual accumulation of waters from any source that may be associated with undesirable effects to life and property” (USEOP, 1994). The latter definition emphasizes certain critical aspects of flooding such as inundation and damage. These aspects will be defined and highlighted significantly throughout this report.

Despite the differing opinions of defining floods or a flood condition, the causes of flooding and methods to control the consequences of flooding have been and remain universal. Causes of the two major types of flooding (river and estuarine/coastal flooding) as well as factors that may intensify floods are summarized in Figure 3.2.

Flood control can be defined as any means used to reduce or alleviate the negative consequences of flooding. The term flood control is often seen as a misnomer due to the fact that the true causes of floods are not controllable. These causes, such as rain or
Figure 3.2: Flood Causes and Intensifying Factors (Smith and Ward, 1998)

snowmelt, are naturally occurring and cannot be modified or altered by anthropogenic means. In addition, some of the above listed flood-intensifying factors, such as basin shape and size, topology, slope, and soil type, cannot be sufficiently altered to have a noticeable effect on flood conditions. Some watershed factors including land use, vegetation, channel configuration and drainage networks can be described as flood-modifying factors which are easily altered by anthropogenic means including urbanization and development, construction, deforestation, and others.

For river floods, flood control is a singular subset of a much larger topic known as floodplain management. Floodplain management, often incorrectly used interchangeably with flood control, is an overall decision-making process whose goal is to achieve appropriate use of floodplains. Appropriate use is any activity that is compatible with the
risk to natural and human resources located in the floodplain, either naturally or by choice. It is the operation of an overall program of corrective and preventive measures for reducing flood damage, including but not limited to watershed management practices, emergency preparedness plans, floodplain management regulations as well as flood control works. Floodplain management incorporates many of the political, social and economic factors that directly affect the severity of floods with respect to damage and the choices involving what methods to employ in limiting this damage (USEOP, 1994).

Any general flood control approach can be described as fitting into one of the following categories: structural, nonstructural, or combination (Mays, 1996). Methods that seek to modify flood runoff through the creation and implementation of engineered structures are classified as structural. Examples of structural methods include channel modifications (channelization), reservoirs, diversions, and levees or dikes. Those methods that seek to modify the damage susceptibility of developed regions within a floodplain are classified as nonstructural. Nonstructural measures serve to adjust the use of flood-prone lands to the flood hazard by a variety of means including: floodplain land acquisition (also commonly referred to as voluntary relocation or property buyouts), land use restrictions (zoning), flood proofing, flood warning systems, drainage maintenance programs, and public awareness or information programs. Each approach has associated advantages and disadvantages. It is beyond the scope of this study to discuss each type of structural or non-structural method. However, due to their potential applications in Clear Creek, channelization and property buyouts will be addressed in some detail.
3.2.1. Structural Methods of Flood Control

The United States has pursued, throughout much of its history, the mostly singular course of attempting to control floods through the use of structures (Reisner, 1993; Smith and Ward, 1998). The structural approach’s rise to dominance began in the 1930s with the passage of the 1936 Flood Control Act and the creation of the Federal Crop Insurance Corporation. This action placed the Federal government in the lead with respect to the Nation’s efforts to control flooding as well as the compensation of its victims. By 1940, the government had assumed the full cost of building and maintaining dams, channel modifications and rectification projects for all navigable waters of the United States (USEOP, 1994). Advances in the fields of fluid mechanics, hydrologic systems, statistical hydrology, evaporation analysis, flood routing, and operations research throughout the decade made the goal of designing and constructing large-scale control structures well within reach and very extremely effective (Bedient and Huber, 1992).

Structural alternatives include channel modifications in a variety of forms: the use of reservoirs or detention/retention ponds to retard or contain flood flows, the construction of levees or floodwalls to contain flows within a designated floodway, and diversions to redirect flood flows to an alternate, off-channel floodway or detention pond. Each of these methods is viable under certain conditions and has associated advantages and disadvantages (Mays, 2001). A brief description of the types of structural alternatives that have been considered for application in the Clear Creek watershed is included in the following section.

3.2.1.1. Flood Control Reservoirs
Flood control reservoirs provide an effective means of managing and controlling flood flows. These basins can range from as simple a structure as a small pond planned alongside a highway or road culvert to a large reservoir with control structures designed to hold extreme volumes of water. Regardless of their size, all reservoirs serve the same function of providing additional storage to attenuate the peak flow experienced during a storm event (Mays, 2001).

Flood control reservoirs can be divided into two basic types, detention and retention. Detention reservoirs or ponds are designed to hold runoff for a short period of time before releasing it to the natural watercourse. Retention ponds are designed to hold runoff for extended periods of time, usually for aesthetic, agricultural, consumptive, or other uses (Chow et al., 1988). Extremely large retention structures are also referred to as storage reservoirs. These storage facilities are associated with large dams and are often not created solely for flood protection but also for water supply. Storage reservoirs are therefore not discussed in this report. Additionally, these types of reservoirs share entirely different hydraulic and environmental impact characteristics than smaller retention basins.

### 3.2.1.1. Advantages and Disadvantages of Flood Control Reservoirs

Reservoir structures have significant advantages and disadvantages as flood control mechanisms. A reservoir type structure has been identified as particularly well suited for damage reduction by the Corps when damageable property is spread over a large geographical area downstream of the reservoir site and when sufficient real estate is
available for its location at reasonable economic, environmental and social cost (USACE, 1996).

The beneficial reduction of peak runoffs and resulting flood protection afforded at a particular point on a stream by a reservoir depends primarily on the fraction of the watershed area above the protected point, which is governed by the location of the reservoir structure. This reduction is progressively reduced as runoff from the watershed area below the reservoir becomes appreciable and the distance downstream of the reservoir increases. This distance varies and is dependent upon watershed characteristics such as the number and size of tributaries and shape of the watershed. To be effective, a reservoir site must be available not too far upstream. This may serve as a significant advantage over other types of flood control since a smaller reservoir may be constructed just upstream of a flood prone location, providing adequate flood protection while limiting excavation costs and environmental impact.

However, this characteristic may serve as a disadvantage when large-scale protection is desired and location sites are limited. Extremely large reservoirs are required to provide significant downstream protection. The reservoir option provides particularly limited protection against large storm events in areas where both the overland slope and natural channel are small (USACE, 1996). Given Clear Creek’s naturally small conveyance and the flat terrain throughout the watershed, this fact weighs heavily upon the selection of detention and retention basins in that area (Dunbar, 2000). Other significant drawbacks include finding locations to dispose of excavated material and aesthetic concerns when the constructed pond is not being used for flood control.
3.2.1.1.2. Application of Flood Control Reservoirs in the Clear Creek Watershed

The use of reservoir storage as a singular flood control option was investigated during the HCFCO restudy discussed earlier. The study determined that this alternative, used alone, would cost approximately $500 million. This figure, although preliminary, dwarfed the original Corps plan cost estimate of $129 million, resulting in the obvious dismissal of this alternative as a feasible option. However, the study also recognized that smaller, localized detention basins, both on-site and downstream, should play an integral role in any agreed upon flood control plan based on their proven flood protection and associated, minimal environmental impact.

3.2.1.2. Channel Modifications

Channel modifications are performed to improve the conveyance characteristics and carrying capacity of a natural streambed. A natural channel’s conveyance and capacity can be improved by altering one or more of the dependent hydraulic variables of slope, depth, width, and roughness (Brookes, 1989). The resulting increased hydraulic efficiency of the “improved” channel results in increased flow velocities, which, in turn results in reduced flood stages or water surface elevations for a given storm event.

These modifications are often referred to as “channelization” and were extensively used throughout the United States in the 20th century with effective results (Smith and Ward, 1998). This technique is especially common as a flood control measure for meandering streams of the South and Southeastern Coastal Plains (Brookes, 1998). However, the channelization approach has always been clouded in controversy as to whether the associated harmful effects on the environment may exceed benefits.
Channelization permits the control of natural, existing waterways, usually to permit or promote economic development or to protect already established urban, agricultural and industrial developments. Specific channelization projects may be undertaken for one or more of a number of reasons: for flood control, to drain wetlands, improve navigation, and to prevent bank erosion and channel migration, thereby protecting neighboring property (USACE, 1996). A history of proven results seemed to make channelization the alternative of choice for many flood control agencies until the mid-1970's. However, as discussed earlier, significant concerns over the environmental impacts associated with channelization and questions over the long-term effectiveness it provides have limited its application in recent years.

3.2.1.2.1. Manning's Equation for Open-Channel Flow and the Advantages Associated with Channelization

Manning’s equation, an empirical uniform flow equation frequently used for open channel flow computations, serves to illustrate how the channelization techniques discussed above result in increased flow capacities for drainage channels. A general understanding of this equation and its variables provides insight as to one of the reasons why channelization is so effective and is a popular choice for flood damage reduction from a hydraulic perspective.

Manning’s equation was presented in 1890 by Robert Manning and relates volumetric flow rate in an open channel (or pipe) to the channel’s conveyance and slope. The equation is as follows:

\[ Q = K S_f^{1/2} \]
which relates the following variables:

\[ Q = \text{Volumetric flow rate (m/sec)} \]

\[ S_r = \text{Energy gradient (non-dimensional)} \]

\[ K = \text{Channel conveyance} \]

The channel conveyance, \( K \), can be further described using the following equation:

\[ K = \left(\frac{1}{n}\right) A R^{2/3} \]

where: \( n = \) Manning’s non-dimensional roughness coefficient which is related to the friction loss associated with water flowing over the bottom and sides of the channel

\( A = \text{Cross-sectional area of the channel (m}^2\) \)

\( R = \text{Hydraulic radius, which is equal to the cross-sectional area divided by the wetted perimeter of the channel (m)} \)

(Note: This empirical formula requires a conversion coefficient equal to 1.49 if used with English units.)

A dimensional analysis of the roughness coefficient, known as Manning’s \( n \), shows units of TL\(^{-1/3}\) and illustrates a theoretical problem with the equation. Additionally, the coefficient plays a critical role in the formula’s accuracy. However, this has not restricted its usefulness and the equation is widely used to solve a variety of open channel flow problems (Bedient and Huber, 1992).

Manning’s equation provides an excellent understanding of how channelization provides flood protection through a variety of processes including increasing the river
bed slope, widening and deepening of the channel to increase its cross-sectional area, realigning or straightening of the channel to minimize the backwater effects of river bends, and the clearing of obstructions to flow along the channel’s banks. Increasing the slope of the channel bed results in an increased energy gradient (which is equal to the channel bed slope under the conditions of uniform flow). Widening and deepening of a natural channel cross-section increases the cross-sectional area, A, and directly permits increased flows. Channel deepening can be achieved either by narrowing the natural channel or by dredging. However, deepening may result in unstable channel banks. Additionally, the deepened channel bed must be matched with existing bed conditions at the upstream and downstream ends of channel modifications. On the other hand, channel widening avoids the problems of bank instability associated with channel deepening while obtaining similar results. However, the increased land required on either side of the river banks must be available or acquired. Depending on the cross-sectional shape after modification (ie. rectangular, trapezoidal, triangular), the hydraulic radius may be increased, further increasing flow capacity. The trapezoidal shape is the most common throughout the United States since it provides increased hydraulic efficiency while permitting increased bank stability as opposed to a rectangular cross-section (Bedient and Huber, 1992). Lastly, the smoothing and straightening of a channel and its banks results in a significantly reduced Manning’s coefficient. Further discussion regarding Manning’s equation and floodplain hydraulics is available in many standard hydrologic and hydraulic texts which discuss these issues from a computer modeling perspective such as Hoggan, 1997.
The various processes involved in channelization serve to make this flood control option extremely effective from a purely hydrologic and/or hydraulic point of view. Channelization has brought major benefits, both direct and indirect, to agriculture, transportation and other sectors of the economy by providing effective flood control throughout the stretches of river reach it has been applied. Its effectiveness is highlighted by the fact that from 1820 to 1970, more than 200,000 miles of the nation's waterways were modified (Schoof, 1980). A. D. Little's report to the Council on Environmental Quality in 1973 stated that the direct benefits of the channelization projects it studied were, in fact, conservatively estimated. It concluded that the vast majority of these flood channel modifications were performing as designed and that the approximate $15 billion invested, to date, nationwide in channel modifications for flood protection were reducing annual damages by $1 billion. Some however, were under-designed and failed to provide adequate flood control effectiveness for their original design storm, such as the Brays and White Oak bayous (Hoblit et al., 1999). Other channel modifications were severely over-designed, ill-conceived or otherwise not properly planned and engineered so that the significant environmental impacts now well-known to accompany them far outweighed the flood control benefits they provided.

Growing concerns over the long-term effectiveness of channelization and the environmental impacts generated by this approach brought the widespread construction of these types of projects to a near standstill. The channelization work of the Soil Conservation Service (SCS) and the Corps became so controversial during the late 1960's that the U.S. Congress commissioned Arthur D. Little Inc. to make a comprehensive nationwide survey of the environmental effects of these projects and report their findings
to the Council on Environmental Quality. This landmark report would serve as the basis for heated debate over the best course of action for flood control throughout the next couple of decades; however, there can be no denying the near immediate and broad-reaching impact this report would have on channelization projects.

3.2.1.2.2. Undesirable Effects of Channelization: Well Documented Concerns

The Little report indicated there are many effects of channel work that must be carefully considered before implementing any channelization scheme. These included:

- Drainage of neighboring wetlands;
- Cutting off of oxbows and stream meanders;
- Lowering of ground water levels resulting in reduced stream recharge;
- Clearing of floodplain bottom land hardwoods and destruction of wildlife habitat;
- Increased erosion of stream banks leading to stability problems;
- Increased downstream deposition of silt and sedimentation;
- Increased downstream flooding as a result of increased upstream velocities;
- Impacts on the stream’s and receiving body’s aquatic life;
- Reduced aesthetics / loss of visual amenity.

The report investigated 42 projects nationwide and is still the most comprehensive study of the effects of channelization to date. The 42 projects encompassed channel works ranging from as small as 1.2 channel miles to as large as 972 channel miles. Some of the projects reviewed showed none of the above effects. The report ranked the above listed effects in three varying degrees of severity ranging from relatively minor significance to relatively pronounced significance. A middle classification of uncertain significance was included when there was insufficient data or substantial debate regarding the significance of the effect. The report summary tables clearly show that project size had significant impact on whether or not the above effects were present. While reduced aesthetics was
of relatively pronounced significance across the range of project sizes, other effects were not dominant until the project size reached the 30 to 40-channel mile range. Thirteen reviewed projects were greater than 35 channel miles in length. Of these, 11 had at least three of the above effects listed as relatively pronounced.

The Little report's findings were sufficient to shift the focus of further research away from whether or not environmental impacts were indeed caused by channelization and toward the development of improved science and technology to determine the extent of those impacts. Through the next two decades, scientific research concerning the impacts of channelization ranging from hydraulic to environmental appeared in a wide variety of publications. These publications varied from international periodicals to the more obscure journals and reports of local environmental and wildlife organizations. Additional areas of concern have been added to the original list, such as increased temperatures due to canopy removal (Livari, 1993) and the pivotal role of channel maintenance in maintaining flood control effectiveness (Brookes, 1988).

Addressing the problem of a lack of summarized research regarding the effects of channelization, Andrew Brookes published one of the most comprehensive texts in the field in 1988. The text, entitled *Channelized Rivers: Perspectives for Environmental Management*, provides an excellent overview of the effects of channelization, both in Britain and the United States. Physical, biological and downstream consequences are discussed in detail. Brookes provides revised construction and maintenance procedures that seek to minimize environmental impact of both newly constructed channels as well as enhancement and restoration techniques for already existing channels. Other
important articles and texts concerned with the various impacts of channelization are presented in Table 3-1.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Important Findings</th>
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</thead>
<tbody>
<tr>
<td>Campbell et al., (1972) and Shankman and Pugh (1992)</td>
<td>Case studies of channelization impacts on a storm hydrograph. Suggests that straightening and increasing channel gradients results in a more efficient flow of water through the channel providing adequate flood control effectiveness for the upstream region of the channelized section only. Sufficiently increased peak flows in the downstream reaches of the channelized section lead to downstream flooding. Sites the importance of a significantly large receiving body of water immediately downstream of a channelized reach to provide sufficient storage for these increased flows.</td>
</tr>
<tr>
<td>Schoof (1980)</td>
<td>Investigated six additional case studies and verified many of the environmental impacts mentioned in the Little report. The paper highlighted the detrimental effects to aquatic habitat (referencing Marzolf, 1978) and riparian habitat (referencing Ferguson, et al., 1975 and Preltwitz, 1976). Additional objections to channelization were raised by the author including concerns of post-channelization development up to the channel banks that could result in devastating damages and the limited utility of channelization unless an appropriately large receiving body of water is present.</td>
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<tr>
<td>Shields and Copeland (1990)</td>
<td>Claims that although channelization has taken a &quot;back seat&quot; to other national issues such as global warming, hazardous waste, etc., it remains a prominent issue at the state and local level. Provides evidence that environmental features (such as complex cross-sections), can be coupled with standard channelization techniques without minimizing hydraulic efficiency. States that continued improvement in the precision (spatial and temporal resolution) of simulation tools and the integration of a variety of professional fields is important for continued improvement toward the environmental design of channels.</td>
</tr>
<tr>
<td>Shankman and Samson (1991)</td>
<td>An important study designed to determine if channelization had significantly reduced the depth and duration of flooding along the Obion River in western Tennessee. Particular importance was focused on suspected differences between upstream and downstream effects. Results revealed that while upstream flow efficiencies were sufficiently increased and floodplain storage sufficiently decreased to reduce flooding in that region, the increased flow velocities resulted in upstream erosion and downstream deposition of sediments. This caused reduced cross-sectional areas just downstream of the channelized region resulting in increased downstream flooding. This result was in contrast to much earlier findings (see Campbell above) that contributed downstream flooding in channelized sections solely to increased flows. In fact, mean annual peak flows on the lower Obion River were approximately the same before and after channelization.</td>
</tr>
<tr>
<td>Livari (1993)</td>
<td>Comparison of field data shows that channels with no extensive removal of canopy did not experience any significant change in average monthly water temperature while those with extensive removal increased an average of 2-5 degrees F through the early summer months.</td>
</tr>
<tr>
<td>Mattingly el al. (1993)</td>
<td>A unique study suggesting that natural stream channel altering projects are far more numerous than indicated by an operating permitting system in the state of Illinois. The study is important nationally because of the widespread occurrence of unauthorized channelization projects accomplished both by private citizens and local governments.</td>
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3.2.1.3. Other Structural Methods

Table 3-2 briefly summarizes other commonly applied structural methods.

<table>
<thead>
<tr>
<th>Structural Method</th>
<th>Description</th>
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<tbody>
<tr>
<td>Diversions</td>
<td>This method is used to reroute or bypass a portion of the flood flow away from a floodprone area. This results in reduced peak flows at the area of interest. The method only provides downstream protection and the amount of protection diminishes downstream in a similar fashion to reservoirs. Diversions are often used in conjunction with off-line (away from the main channel) reservoirs and by-pass channels as the diverted flows must still be contained and/or passed around the area of concern. Their significant drawback is that sufficient real estate must be available for whatever mechanism is used to contain or accommodate the diverted flows.</td>
</tr>
<tr>
<td>By-Pass Channels</td>
<td>Man-made channels used to contain and transport diverted storm flows away from or around a significantly floodprone or ecologically sensitive area. These channels are subject to the same limitations as channel improvements to the main stream or floodway. Significant right-of-way and property acquisitions hinder this method's applicability. Additionally, there are usually significant excavation costs associated with creating channels where there are no currently existing streambeds. The Harris County Flood Control District considered this alternative for application in Clear Creek.</td>
</tr>
<tr>
<td>Levees / Floodwalls / Dikes</td>
<td>Linear structures, often earthen embankments or reinforced concrete walls, built parallel to the main stream with the purpose of containing a specified design overbank flow. These structures are often located very near the natural floodway and on both sides of the river, providing protection to the entire natural floodplain. This alternative is considered the oldest and most widespread type of localized flood defense worldwide (Starosolszky, 1994) and plays a fundamental role in the life and work of many countries, including the United States. Levees increase the local carrying capacity of the channel and are intended to prevent all flood damage to the adjacent river corridor until the water level exceeds the top of the structure. Their most significant drawback is that if overtopped, flood damage is incurred as if the structure did not exist. Given the historical trend of development within the floodplain (especially with the additional sense of security provided by the floodwalls), damages often exceed what would have occurred without the levee. Additional drawbacks include increased downstream flow rates and resulting downstream flood damages, significant right-of-way requirements, notable environmental impacts similar to channelization and others (NWF, 1998).</td>
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3.2.1.4. The "Astonishing Paradox"
Despite the expenditure of billions of dollars in structural controls, flood damages throughout the United States have continued to rise. In 1969, near the height of the Corps' and Soil Conservation Service's construction of flood control mechanisms, the U.S. Water Resources Council reported nationwide flood-damage reduction benefits at about $1.0 billion annually in response to $6.1 billion of expenditures toward that effort (A.D. Little, 1973). However, the Council also reported that based on current trends of land-use and development, the total annual flood damage potential for the Nation was anticipated to increase from $1.7 billion in 1966 to $5.0 billion in the year 2020 (A.D. Little, 1973). As of 1998, annual U.S. flood damages had already exceeded $4 billion (NWF, 1998). Figure 3.3 illustrates the steady increase in flood loss damages in the U.S.

![Historical Flood Losses](image)

**Figure 3.3: Historical Flood Losses in the United States (NWF, 1998)**
through the 20th century (NWF, 1998).

It must be emphasized that one of the most undesirable aspects of structural devices is that they are expected to fail if an occurrence exceeds their design specifications. In designing protective works, there are several names for the expected flood a structure should protect against, such as the “design flood or storm”, the “regional flood”, the “standard project flood”, and the “maximum probable flood”. The very existence of these protective structures has given rise to an astonishing paradox of increasing flood damages despite the enormous expenditures to reduce them. John Wilkinson, in a report written to the Federal Reserve Bank of Boston in 1967, attempted to summarize factors creating this circumstance.

“Memories are short; upstream detention storage and flood walls foster a false sense of security; flood-prone urban land has high locational value; development near the river channel constricts flood passage, creating backwater over a flood plain on which development also encroaches; developers don’t pay for flood relief, rehabilitation or prevention and therefore encounter no financial disincentives to not develop in the expanding floodplain; the next flood produces even greater damage; this in turn justifies more protection; after a brief period, confidence is regained and more development is encouraged” (A.D. Little, 1973).

Fortunately, ways to break this cycle have been understood since before the 1960’s. Unfortunately, several technological, political and economic barriers have hindered their promulgation and acceptance.

3.2.2. Non-Structural Methods

Non-structural measures are used to reduce the damage potential of a structure or facility within the floodplain by adjusting the use of flood-prone lands to the flood
hazard. This approach is in stark contrast to its structural counterpart, which instead attempt to minimize the flood hazard by actually altering flood flows (Mays, 2001). The amounts of reduction in damage potential achieved by non-structural measures vary significantly, depending on the mechanism implemented.

There exist a wide variety of non-structural flood controls, including: flood proofing, flood warning mechanisms, land-use controls such as zoning and development ordinances, flood insurance programs, flood preparedness activities, public awareness and education programs, existing primary and secondary drainage system maintenance, and the acquisition of floodplain land or voluntary flood-prone property buyouts (USEOP, 1994). Each of these particular methods has associated advantages and disadvantages.

3.2.2.1. History of Non-implementation

Non-structural methods have only recently been given serious consideration as a primary means of reducing flood damages, despite the fact that their potential as such has been researched for over 50 years. Alternative options to structural methods were perhaps first articulated by White in 1945. In the 1950's, the Tennessee Valley Authority, with its broad mandate for water-management experimentation, proved an effective pilot program for many non-structural measures with promising results. Additionally, the non-structural movement was somewhat popularized by Leopold and Maddock in 1954 as well as Hoyt and Langbein in 1955, resulting in professional interest and the involvement of the American Society of Civil Engineers in the development of a Guide for the Development of Flood Plain Regulations. However, it was 1966 before a
concerted effort to integrate non-structural measures into federal flood control policy would take place.

A 1966 Presidential Task Force on Floodplain Regulations, charged with making recommendations for improving the existing federal flood control policy, highlighted five basic objectives for developing a unified national program for managing flood losses. These were:

- Improve the basic knowledge about flood hazards at both the local and regional level;
- Improve planning and coordinating of all new developments in the floodplain;
- Improve technical services available to floodplain managers;
- Create a national program for flood insurance;
- Allow for the future adjustment of federal flood control policy to changing needs.

The Task Force report spurred action to develop floodplain management strategies amongst a myriad of federal agencies, including the Soil Conservation Service and the Corps. Additionally, one of the most important results of the report was the creation of the National Flood Insurance Program (NFIP) through the Flood Control Act of 1968.

3.2.2.2. The National Flood Insurance Program (NFIP)

Despite substantial investments in flood control measures, flood losses continued to rise through the early 1960’s. This, combined with a burgeoning environmental ethic, set the stage for a new approach to floodplain management. In response to the failure on
the part of the private sector to provide reasonable, cost-effective flood insurance to the average buyer, the National Flood Insurance Program (NFIP) was created by the Flood Insurance Act of 1968 (NWF, 1998).

This program was designed to provide cost-effective insurance to property owners who live within the 100-year floodplain in communities that have established a minimum standard of hazard mitigation provisions. The goal, in principal, was simple – allow the insurance premiums and not the general taxpayer to cover the cost of flood damages. The required mitigation provisions would enforce floodplain development regulations, guidelines and limitations. The NFIP was placed under the cognizance of the newly formed Federal Emergency Management Agency (FEMA) in 1979.

The program has enjoyed some success. Flood insurance premiums covered $10.4 billion in losses and program expenses between the years 1977 and 1997 (NWF, 1998). As an additional benefit, the program has stimulated additional floodplain management concepts and programs throughout the country.

However, the program has experienced major shortcomings in the form of policy and financial distortions, repetitive loss properties, poor enforcement of regulations pivotal to the success of the program, and significant inaccuracies in floodplain delineations (NWF, 1998). Many opponents of the plan argue that it has actually encouraged development and rebuilding in high-risk areas with the promise of a “limitless guarantee” of governmentally sponsored bailouts.

The existence of repetitive loss properties (RLPs) is a drain to the efficiency and effectiveness of the program. A repetitive loss property is any insured property that has sustained two or more flood losses of at least $1,000 each in any 10-year period. (NWF,
Although only comprising two percent of the properties insured, these properties are responsible for nearly 40 percent of the program's payments. The city of Houston is ranked number three on the Top 200 Repetitive Loss Communities Rankings by payments. The city has an estimated 2,030 RLPs with total payments in excess of $114 million. The city of Friendswood is ranked number ten on the same list, with an estimated 314 RLPs with total payments in excess of $29 million (NWF, 1998).

Despite this evidence that the NFIP is not the singular flood control answer, its advantages and effectiveness are far reaching. The program has spurred an ever-increasing awareness of the benefits of a sound flood mitigation strategy and even highlighted the environmental benefits of the floodplain when put to its natural use.

3.2.2.3. Benefits of the Natural Floodplain

One of the most beneficial aspects of a non-structural approach to flood control is its greater utilization of the benefits provided by the natural floodplain. Floodplains are the lowlands adjoining the channels of rivers, streams or other watercourses, or the shorelines of oceans, lakes, or other bodies of standing water (FIMTF, 1992). The "loss" or development of a floodplain results in significant losses in water, biological, and human resources that may easily outweigh the property value of the floodplain. Water resources include a natural means of flood and erosion control that reduces sedimentation, provides large amounts of storage, and naturally reduces flood peaks and valleys. With respect to water quality, the floodplain can filter nutrients and impurities from runoff prior to deposition in the river and serves as a natural temperature and organic waste moderator. From the biological perspective, floodplains increase
biological productivity and provide important fish and wildlife habitats. The integrity of the existing ecosystem is maintained by a sustained biodiversity and an atmosphere conducive to plant and animal growth. Often, these areas serve as natural habitats or wildlife corridors for endangered species. Floodplains represent a wealth of human resources by providing prime locations for aquaculture, enhanced agricultural lands, open space that can be used for public recreation or nature appreciation, and they are often rich in cultural resources as well.

Nevertheless, it is a difficult task to economically quantify the total value of floodplains and other environmental resources. Accurately valuing environmental amenities has been the subject of recent debate (NRC, 1999a). It is generally agreed that an established standard for assigning a dollar-cost or worth to specific amenities such as wetlands, bottomland hardwoods, riparian forest, green space, etc., would provide a much needed quantification of the indirect benefits provided by non-structural measures. However, there are often too many widely differing variables from location to location to make this improbable. Nevertheless, a variety of methods to measure environmental costs and benefits have been established such as contingent valuation, the benefit-transfer method and hedonic pricing. Specifics on these approaches as well as others are available in a 1999 National Research Council publication entitled New Directions for Water Resources: Planning for the U.S. Army Corps of Engineers.

3.2.2.4. Voluntary Buyouts of Flood-prone Property

The buyout of high-risk floodplain properties from willing sellers and the relocation of at-risk buildings and structures out of the floodplain are non-structural
alternatives that have received increased attention in recent years (NWF, 1998). The catastrophic Midwest flood of 1993, as well as several resulting post-flood studies and recommendations, spurred national interest in this particular alternative. The essentials of the voluntary buyout program are clearly stated in the National Wildlife Federation report entitled “Higher Ground: A Report on Voluntary Property Buyouts in the Nation’s Floodplains – A Common Ground Solution Serving People at Risk, Taxpayers and the Environment.” The pros and cons of this flood control alternative will be briefly summarized.

The overall goal of this alternative is simply achieved in theory but very difficult to achieve in practice. Theoretically, a one-time expenditure of public funds would result in the evacuation of a specified floodplain area. This cost, although significant, would result in the complete elimination of flood risk for those areas. Ideally, the long-term financial benefits (in the form of reduced flood losses and an increase in environmental benefits) would more than account for the initial cost. However, there are a number of factors that have complicated the application of this option.

Higher Ground lists the ideal goals of a well-planned voluntary buyout program. These include:

- Seek and secure a combination of federal, state and local funds for one-time buyouts of high-risk properties from willing sellers.
- Ensure the return of the purchased property to its natural floodplain state
- Pass legislation that would prohibit the expenditure of any future disaster assistance to that location; and
• Provide assistance to former property owners and tenants to move to higher ground and out of harm's way, and, as appropriate, relocate homes and businesses outside the floodplain.

This type of program has specific advantages over structural flood control options as well as many other non-structural methods. Perhaps its strongest point is that it provides permanent disaster relief and help to at-risk people and property. Other strong points include the fact that proper implementation of this plan would end an unlimited disaster relief obligation on public funds by putting tax dollars to the most cost-effective use possible. Lastly, the program would truly realize the environmental benefits of the floodplains by restoring them to their natural ecological functions and thereby provide many of the benefits listed earlier.

Buyouts are not without limitations and drawbacks. Buyout programs can reduce the local tax base by forcing businesses and significant investors to "look elsewhere" – namely other communities. Improper regulatory enforcement or a lack of a sound land management plan after the buyouts may result in renewed development pressure and force a change in local regulatory laws. Many buyout plans have been criticized as a method of driving low-income residents out of a central, often desirable, part of a community, paving the way for future up-scale developments. Lastly, significant costs, time and careful planning are required to successfully implement a buyout plan, regardless of its scope. The fact that buyouts involve significant changes in land use patterns often within well-established areas of a community often present insurmountable
legal, social and economic difficulties that force the decision making process toward a more conventional structural method (NWF, 1998).

3.2.2.5. A Changing Trend Toward the Implementation of Non-Structural Measures

Recently, several factors have combined to make the use of non-structural methods critical to any sound, floodplain management or flood control scheme. These factors range from policy changes, both at the federal and local levels, to recent technological advancements, which have enabled indirect modeling of certain non-structural measures.

The most significant changes in policy affecting non-structural implementation come by way of the Water Resource Development Acts (WRDAs) of 1996 and 1999. Section 575 of the 1996 WRDA has altered the cost-benefit ratio, a ratio used by the Corps to determine the economic viability of undertaking a specified project, so that non-structural projects achieve a much more favorable ratio (in terms of benefit per cost) than before. Specifically, the 1996 WRDA, subsequently amended in 1999, prevents the consideration of benefits from existing non-federal flood control structures or non-structural projects in calculating the economic costs and benefits of the project as a whole. Because the impact of a proposed project will now be measured against a blank slate of zero benefits (as opposed to previously counting the vastly structural measures already in place as a benefit to the new project), non-structural proposals are finally being considered “on the same playing field” as their structural alternatives.

Many recent publications, including the report of the Interagency Floodplain management Review Committee entitled Sharing the Challenge: Floodplain
Management into the 21st century and the National Research Council's New Directions in Water Resources Planning mentioned earlier, have prompted new approaches toward and evaluations of existing flood control schemes. Federal agencies, such as the Corps, have responded to conclusions reached by the above reports. In fact, the Corps has recently established the Flood Mitigation and Riverine Restoration Program, more commonly known as Challenge 21. The program will focus on non-structural solutions to reducing flood damages, while maintaining the flexibility to use more traditional structures where appropriate. Under this program, the Corps will pay 65 percent of the cost of buying properties in the floodplain and restore the floodplain ecosystem to its pristine state. Communities participating in the program will pay the remaining 35 percent. Although the program is in its infancy and currently funded with only $200 million over the next six years, it represents a significant change in the overall flood-control mission of the Corps (Shoup, 2000).

Lastly, recent technological advancements in hydrologic science and geographic information systems (GIS) have played an instrumental role in allowing quicker, less costly, and more accurate floodplain analysis. This, in turn, has allowed for improved modeling of various structural and non-structural alternative combinations. These advancements have served to answer some of the many other reasons why non-structural measures have not been fully implemented. A brief discussion of two of these significant advancements, GIS and Next Generation Radar (NEXRAD) follows.

3.3. Next Generation Radar (NEXRAD)
Radar has become a rapidly growing source of spatially and temporally
distributed rainfall data ideal for hydrologic modeling. A vast network of next generation
WSR-88D (NEXRAD) doppler radars, deployed by the National Weather Service, results
in nationwide radar coverage providing the capability of obtaining accurate estimations
of rainfall intensity over nearly any area within the continental U.S. Although radar
technology itself is not new, the application of this technology toward hydrological
purposes such as accurate hydrologic modeling and real-time flood alert systems is
relatively recent (Bedient et al., 2000).

Rainfall data are traditionally obtained from an often sparse network of rain
gauges that may not record a given storm event's widely variable spatial distribution and
intensity. These errors arise from the fact that rain gauges are only capable of sampling
rain at distinct points. Depending on the density of the network, rainfall patterns may
pass between gages, resulting in an underestimation of the actual total rainfall. This is
especially true in regions, such as Houston, that experience highly variable convective
storm events. Recent interest in using radar estimates of rainfall in hydrologic modeling
has risen from the desire to reduce this error by utilizing the radar's capability of
providing complete coverage within the area of interest (Vieux and Bedient, 1998).

3.3.1. History of the NEXRAD (WSR-88D) Radar

The WSR-88D (Weather Surveillance Radar 1988, Doppler) radar, commonly
referred to as NEXRAD, was developed to replace pre-Doppler technology radars for the
purpose of providing an advanced early warning system for tornadoes. The new Doppler
technology allows meteorologists to detect specific circulation patterns within a storm
that generally precede the touchdown of tornadoes. The first prototype system was installed in Norman, Oklahoma, in 1988 (Knox and Canter, 1991). The first full-scale WSR-88D radar was deployed in 1992 (Fulton et al., 1998).

The geographic location of each radar within the network was optimized in order to provide full nationwide coverage as well as to provide effective coverage for a variety of meteorological events at different areas. For example, the NEXRAD radar covering the Houston area was located south of the metropolitan area in order to “ideally spot frontal storms from the north to the southwest, severe thunderstorms from the west; as well as tropical storms and hurricanes from the southeast to east” (Hoblit et al., 1999).

The NEXRAD radar network currently has a number of wide ranging applications within the water resource field. Some of these include: flood forecasting and early warning systems, improved precipitation estimations, long-term water balance studies at the basin scale, as well as more localized, short-term flood prediction (Hoblit et al., 1999).

3.3.2. System Characteristics

The NEXRAD doppler radar is a 10 cm wavelength or S-band transmitter that records reflectivity, radial velocity, and the spectrum width of reflected signals. Successive radar tilt angles are employed to cover the entire volume of the atmosphere out to 460 km for reflectivity, 230 km for precipitation, velocity and spectrum width. Radar reflectivity is collected at 1-km range intervals and each 1-degree of radial resolution, producing a radial coordinate system of reflectivities for each tilt angle (Crum and Alberty, 1993).
Using basic relationships between reflectivity (referred to as Z) in units of dBZ and rainfall rate (referred to as R) in mm/hr, the rainfall rate in that 1-km by 1-degree area can be estimated, with a greater reflectivity indicating a heavier rainfall amount. During the various stages of signal processing in the data stream, anomalous reflectivities such as ground clutter are removed. The reflectivity signal is then converted to a rainfall rate using a Z-R relationship.

The particular Z-R relationship used to convert reflectivity to rainfall can have a significant impact on the accuracy of the rainfall estimation. The "standard" Z-R relationship used with the initial installation of all WSR-88D radars was \( Z = 300R^{1.4} \). The National Weather Service has adopted in some cases the "tropical" Z-R relationship (\( Z = 250R^{1.2} \)), which is more representative of warm tropical rainfall drop distributions. Recent research in the Houston area has revealed significantly improved results using the tropical Z-R relationship (Vieux and Bedient, 1998). Nevertheless, the Z-R relationship is often in error, because this empirical relationship depends on the drop-size distribution, which varies throughout a particular storm event.

To overcome the estimation errors inherent in the Z-R relationship, calibration with rain gauges can be performed. This procedure usually consists of comparing accumulations between radar and gauge at a particular gauge location. The ratio of the radar estimation to gauge measurement is termed a bias. A mean field bias consists of comparing many radar/gauge pairs of accumulations and then averaging them to get a mean over some geographic region. For example, if the radar is underestimating by 20%, the rainfall fields are increased by 20% to compensate for the bias. Calibration of the
radar to rain gauge accumulations is the most commonly used technique for correcting radar rainfall estimates.

3.3.3. Hydrologic Modeling Using Radar Estimates

There have been numerous attempts at hydrologic modeling utilizing radar estimated rainfall rates. There are a number of reasons behind the application of radar in this field. Two of the most important include the previously discussed problems associated with rain gage estimation of radar and the overall success of radar at estimating rainfall amounts.

Schell et al. modeled the rainfall over a small (8.13 km²) watershed in Canada. Rainfall data was entered into a simple lumped parameter model both as gage-adjusted radar estimates and as rain gage measured data. Peak flow measurements from the radar performed better than the estimated peak flow using rain gage data in three of the four storms. Overall runoff calculations were also improved by the radar data in three of the four model runs. Schell concluded that calibrated radar rainfall estimates could provide rainfall inputs to a rainfall-runoff model superior to those obtained from a single rain gage.

Vieux and Bedient (1998), Gladwell (1998) and Bedient et al. (2000) all examined the hydrologic response of various watersheds in the Houston area to radar estimated rainfall for one or both of two major storms in the area. Vieux and Bedient showed an accurate hydrologic response in a HEC-1 model program utilizing adjusted NEXRAD radar over the Clear Creek watershed for the October 17-18, 1994 storm event. Gladwell studied both the same October 1994 event and a January 1998 event over the
Brays Bayou watershed utilizing similar methods to Vieux and Bedient. The October 1994 storm was modeled successfully using the tropical Z-R relationship, with an excellent match between the observed and computed outflow hydrographs. Bedient et al. examined three storms over Brays Bayou in Houston again using the tropical Z-R relationship. This data was not adjusted to the rain gages. Nevertheless, another excellent match was obtained between the observed and computed hydrographs using both radar estimated and gage measured rainfall. The April 1997 and January 1998 events revealed major underestimation of the rainfall according to the rain gage data. The radar estimated rainfall model was found to perform significantly better for all three storms.

Other important papers that address previous attempts at utilizing radar rainfall estimates in a hydrologic model are shown in Table 3-3. These references were obtained from Hoblit, 1999.

Table 3-3: Utilizing Radar Rainfall Estimates in Hydrologic Models

<table>
<thead>
<tr>
<th>Reference</th>
<th>Summary of Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td>James et al. (1993)</td>
<td>Modeled the hydrologic response on a 785 mi² watershed in Mississippi for a November 1986 storm using the Texas A&amp;M Watershed Model. The study concluded that the rising limb and peak discharge of all streamflow hydrographs computed from gage-adjusted weather radar data were more accurate than the corresponding hydrographs computed from gage precipitation data alone.</td>
</tr>
<tr>
<td>Mimikou and Baltas (1994)</td>
<td>Discuss the accuracy of radar compared to point rainfall values from gauges for flood forecasting. Utilized the HEC-1 modeling program to study a 2,763 km² watershed in central Greece. The output hydrographs computed from gage-adjusted radar was more accurate than the output hydrographs computed from gage precipitation data alone for six different storms.</td>
</tr>
<tr>
<td>Peters and Easton (1996)</td>
<td>Used the Modified Clark program, a distributed hydrologic method available in the HEC-HMS modeling program, to model three storm events over a 4,163 km² watershed in northwestern Oklahoma and northwestern Arkansas. NEXRAD rainfall data was used as the rainfall input. Runoff hydrographs the four sub-basins and the outflow hydrograph for the entire watershed were computed for the three storms. Each computed hydrograph matched observed flows with a good fit.</td>
</tr>
</tbody>
</table>
3.4. Geographic Information Systems (GIS)

A Geographic Information System (GIS) is a general-purpose computer-based technology that allows for the efficient collection, construction, integration and dissemination of vast amounts of spatially referenced digital data. As technological advancements in computer science have increased the power and capability of personal computers, there has been steadily increasing interest in discovering new applications of GIS (Maidment and Djokic, 2000). Recently, the most successful applications have included data collection and processing, computer modeling, and policy formulation (Singh and Fiorentino, 1996). In particular, a close relationship has developed between GIS and the hydrologic sciences due to the ability of any GIS software to relate various, spatially diverse data sets common to this field. These types of data sets include land use and soil classifications, river reaches, topography, aerial photography, floodplain mapping, etc. GIS can also assist in the design, calibration, modification and comparison of hydrologic and hydraulic models. The integration of GIS with the hydrologic field is spreading and substantial opportunities exist for continued developments.

This section will discuss some of the recent applications of GIS technology in the hydrologic and hydraulic fields. Additionally, it will define essential terminology as well as the different types of data sets utilized in a GIS program.

3.4.1. History of Geographic Information Systems

The advent of geographic information systems was the result of the growing societal need for geographic information, of a change in the technology that made such systems possible, and combined private sector initiative and government foresight that
initiated and eventually sustained their development. Canada in the early 1960s, concerned with growing pressures on its natural resources, developed the first GIS for the management and effective utilization of mapped information originally collected for the Canada Land Inventory. The manual methods of map analysis then prevalent, although accurate and worthy, were highly labor intensive and time consuming. Additionally, there were not enough trained personnel to carry out the analysis of such large amounts of data in that manner. The change from vacuum tubes to transistors in the computer science arena was a critical change in technology that occurred around that time which made computers faster, more reliable, and cheaper. Additionally, these transistors gave computers larger memories so that they could be used for information storage as well as calculations (Foresman, 1998). Roger F. Tomlinson, then employed by a large aerial survey company specializing in high-technology use of photogrammetric, geophysical, and map-making equipment, originated the concept of a GIS when the company was asked by the Canadian government to analyze all available map series in order to propose locations for new forest plantations as well as new pulp and paper mills. The solution to this daunting task was the creation of many maps, in digital form, that were linked together across Canada. This permanent database would be used to find acceptable solutions not only for the original forest/paper mill problem but would become widely available for the relatively quick and precise analysis of many other land use related problems in the future.

Over the next 30 years, various evolutions in GIS technology, often following advances in computer technology such as the personal computer, would serve to create the powerful geographic information systems available today.
3.4.2. Applications of GIS Technology in the Hydrologic and Hydraulic Fields

Several publications concerning the application of GIS technology in the hydrologic and hydraulic fields have been released within the last decade. A detailed discussion of all of the important breakthroughs in applying this technology is beyond the scope of this report. However, the reader is referenced to a publication entitled Hydrologic and Hydraulic Modeling Support with Geographic Information Systems. David Maidment and Dean Djokic edited the publication, released by the Environmental Systems Research Institute in 2000. The publication includes an excellent list of the most recent applications of GIS technology in these fields including Ackerman et al., 1999; Dodson and Li, 1999; Garbrecht and Martz, 1999; and Kraus, 1999.

3.4.3. Some Important Terminology in GIS

For both simplicity and clarification, the following is a very brief list of some of the terminology used in later sections of this report that, although practically common knowledge for persons with even limited GIS experience, may not be readily familiar to others. These definitions were obtained from DeMers, 2000.

**Address Matching:** The process of defining exact addresses and linking them to specific locations along linear objects

**Digital Elevation Models (DEM):** A digital model of land form data represented as point or raster elevation values.
**Digital Orthophotoquads (DOQ or DOQQ if quarter quads):** A digital version of aerial photographs that are constructed to eliminate image displacement due to changes in aircraft tilt and topographic relief.

**Raster Data:** A form of GIS graphic data structure that quantizes space into a series of uniformly shaped cells that contain one data value per cell. Often applied to Digital Elevation Models, Precipitation Arrays and other continuous data surfaces or models.

**Triangulated Irregular Networks (TINs):** A vector data model that uses triangular facets as a means of explicitly storing and displaying surface information. Most often used for elevation models commonly referred to as Digital Terrain Models.

**Vector Data:** A graphic data structure that represents the points, lines, and areas of geographical space by exact X and Y coordinates.
Chapter 4. Analysis Tools and Methodology

The objectives of this study were laid out in Chapter 1. For convenience, these objectives are again listed below:

1. Evaluate the overall flooding problem in the Clear Creek watershed and determine a suitable area of interest within the watershed for detailed study.

2. Utilize NEXRAD rainfall estimates for rainfall input into a hydrologic model of the Clear Creek watershed.

3. Develop and illustrate the utility of a hydrologic model for the Clear Creek watershed in the improved Hydrologic Modeling System (HMS) format.

4. Develop and illustrate the utility of a hydraulic model in the River Analysis System (RAS) format for determining water surface elevations in the study area.

5. Utilize available GIS-data, ArcView and the HEC-GeoRAS software to develop digital floodplains that can be used as both qualitative and quantitative measures of the effectiveness of various flood control options.

6. Combine the above models to investigate the effectiveness of two proposed flood control alternatives including limited channelization and floodplain property buyouts.

These objectives were met by following advanced hydrologic and hydraulic procedures for taking a given input rainfall amount through a series of computations to produce a corresponding floodplain map of the area. This overall methodology is illustrated in Figure 4.1.

Any hydrologic and/or hydraulic analysis must begin with either an actual or theoretical rainfall input (step 1). This rainfall input is converted to runoff via a hydrologic model (HEC-HMS), resulting in a corresponding hydrograph of flow versus time (step 2). The peak flows of each of these hydrographs are used as input to a hydraulic model (HEC-RAS) to calculate the resulting water surface elevations along
Figure 4.1: Schematic of Overall Methodology

particular reaches (step 3). The HEC-GeoRas program then allows for the conversion of these elevations to a unique inundation area known as a floodplain (step 4). Finally, additional GIS technology can be combined with the resulting digital data displays to analyze a wide variety of flood control alternatives in a quick and efficient manner (step 5). This chapter will briefly discuss the methodology involved with the development, calibration and application of the required models to reach each of the above steps.

It is important to mention that while a full watershed-wide hydrologic analysis was completed in this study, the hydraulic analysis portion focused only on a particular area of interest. The criteria for selecting this area of interest are discussed later in this chapter. Narrowing the focus of the hydraulic analysis was done to reduce the financial expenditure associated with data gathering. However, the overall objective of applying
these new technologies within a GIS framework is still met. Additionally, the overall approach described in this research may be readily applied to other areas throughout the watershed after the required digital data is obtained.

A wide variety of computer models and data analysis tools were used in this study. The majority of these tools, such as the HEC-HMS hydrologic modeling program and the HEC-GeoRas GIS extension, are relatively new technologies that have only recently been introduced for widespread use (1998, 1999 releases respectively). Others, such as NEXRAD and HEC-RAS, have been utilized for just a few years. However, the combined application of all of these tools provides a new and unique approach toward floodplain management and flood studies. This combination approach capitalizes on advances in the availability of digital information and the marked increase in computing capabilities at the desktop level experienced over the last decade. Various sections of this chapter contain brief descriptions of the specific modeling programs and data analysis tools utilized in this study prior to the discussion of their actual application.

4.1. Defining the Study Area for Hydrologic and Hydraulic Analysis

Before proceeding with a discussion of the methodology, it is useful to delineate what regions of the Clear Creek watershed were included in both the hydrologic and hydraulic analyses completed for this study.

The hydrologic portion of this work was completed on all subwatersheds draining directly into Clear Creek or any of its tributaries until its outfall into Clear Lake. Subwatersheds were patterned from boundaries established by the Harris County Flood Control District (HCFCD). This resulted in 48 subwatersheds divided amongst Clear
Creek and its major tributaries. The subwatershed delineations, their respective drainage areas and receiving streams are detailed in Figure 4.2 and Table 4-1.

Table 4-1: Drainage Areas for Creeks and Tributaries within the Hydrologic Study Region

<table>
<thead>
<tr>
<th>Creek or Tributary</th>
<th>Drainage Area (mi²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chigger</td>
<td>16.9</td>
</tr>
<tr>
<td>Clear</td>
<td>85.2</td>
</tr>
<tr>
<td>Cowart</td>
<td>20.0</td>
</tr>
<tr>
<td>Hickory</td>
<td>8.1</td>
</tr>
<tr>
<td>Marys</td>
<td>17.2</td>
</tr>
<tr>
<td>Turkey</td>
<td>13.6</td>
</tr>
<tr>
<td>Magnolia</td>
<td>3.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>164.0</td>
</tr>
</tbody>
</table>

Figure 4.2: Hydrologic and Hydraulic Study Areas for Clear Creek
The hydraulic portion of this study focused on a particular 6.5-mile reach of Clear Creek that runs through Friendswood, Texas. The reach extends from just upstream of the Clear Creek and Marys Creek confluence to the Clear Creek and Chigger Creek confluence. This reach was believed to be an excellent location for detailed study for two reasons. First, flood damages through that reach have been disproportionately large compared to other areas around the watershed (NWF, 1998; HCFC, 1997). Second, both structural and non-structural flood control options could easily be implemented in that area with a relatively minimum amount of damage to the riparian forest. This is mainly due to the fact that rapid development within the Friendswood area has already resulted in significant floodplain encroachment. Figure 4.2 also shows the location of the hydraulic study region by highlighting the reach in red.

4.2. Rainfall Data

Both NEXRAD-estimated rainfall data and rain gage data were used as part of the rainfall-runoff analysis for the Clear Creek watershed. A brief description of both forms of data is given below.

4.2.1. Rain Gage Data

All but one of the rain gages used for the hydrologic modeling of the Clear Creek watershed belong to a collection of rain gages known as the Harris County Office of Emergency Management (HCOEM) ALERT network (HCOEM, 2000). The HCOEM ALERT (Automated Evaluation in Real-Time) network is a system of rain gages spread throughout Harris County. Each watershed within the county has, on average, 15 operating gages in or around its area. The number of gages throughout the county has
steadily increased in order to improve the density of the network, thereby reducing the errors associated with the spatial distribution of the gages. The rain gages are of the tipping-bucket style and record the time when a given amount of rainfall is collected by the gage. The data are available in either a real-time format or in archived format from the HCOEM web page at http://www.hcoem.co.harris.tx.us/oem/.

The fact that only about 50% of the Clear Creek watershed lies within the borders of Harris County has presented a data-gathering problem for hydrologists desiring accurate rainfall estimates in that area. As a result of its geographic location, only about half of the watershed was covered by the HCOEM network. In addition, the HCOEM network was less dense in outlying regions of the county such as Clear Creek. Therefore, coverage of the Clear Creek watershed was sparse through the early and mid-1990s. Steady and continued expansion of the number of rain gages in the network has begun to address this problem. Figure 4.3 shows the rain gage locations and resulting Thiessen

![Map of Clear Creek Watershed](image)

**Figure 4.3: Rain Gauge Locations in the Clear Creek Watershed during October 1994**
polygons during the 1994 storm event.

Unfortunately, increasing the number and density of rain gages cannot improve historical rainfall data vital to the accuracy of the analysis of past storms. However, improving the rainfall accuracy and coverage for any historical storm is possible by utilizing archived NEXRAD data.

4.2.2. WSR-88D Next Generation Radar (NEXRAD)

A detailed discussion of NEXRAD technology was presented in Chapter 3 of this report. Therefore, this section will focus on the application of NEXRAD in the Clear Creek watershed and the benefits of connecting this data with other GIS-based information.

4.2.2.1. Applying NEXRAD in the Clear Creek Watershed

There are two primary advantages in applying NEXRAD technology for obtaining rainfall estimates in the Clear Creek watershed. The first is the ability to provide complete, watershed wide coverage (Hoblit et al., 1999). This coverage proves particularly useful in the more poorly monitored southwestern portions of the watershed. The second is the ability to permanently store this data in an archived status for future analysis. This feature has allowed for the repeated and careful analysis of past storm events such as the one experienced in Clear Creek during the dates of October 14-19th, 1994 (Vieux and Bedient, 1998).

The KHGX radar station, located approximately 25 miles south of Houston, is in an ideal geographical position for monitoring rainfall intensity throughout the watershed. The 150-mile range of the station provides complete spatial coverage illustrated in Figure
4.4. In addition, interference in the form of false reflected signals is at a minimum. This is attributable both to the relatively flat topography and the absence of a significant number of man-made structures such as tall buildings located within the line-of-sight of the radar station.

![Figure 4.4: KHGX Radar Location with 50-Mile Range Circles](image)

Perhaps the most important disadvantage of NEXRAD lies in the fact that it provides an estimate of rainfall intensity as opposed to a direct measurement. Attempts to gage-adjust radar rainfall estimates should always be made unless a preliminary hydrologic analysis shows a minimal effect on the hydrographs of interest (Mimikou and Baltas, 1994). Gage adjustments were not performed for the October 1994 storm event based on excellent matches between the observed and modeled outfall hydrographs (Vieux and Bedient, 1998).
4.2.2.2. Combining NEXRAD with other Geographic Information System Data

Due to the spatial orientation of the data provided by radar, a GIS is an excellent display and analysis tool for the vast amounts of data generated by NEXRAD. The capability of overlapping various information layers (as discussed in chapter 3) permits a wide variety of display options. However, the advantages of combining NEXRAD and a GIS go far beyond the display of data. Advanced GIS extensions such as ArcView’s Spatial Analyst allow for the quick and efficient manipulation of rainfall data. Examples of this include performing simple adjustments to rainfall amounts to account for losses and calibrating radar rainfall estimates with available rain gages. Another example includes combining gridded radar rainfall data with a GIS shapefile of a watershed to calculate individual hyetographs (graphs of rainfall versus time) for each particular subwatershed. This procedure was utilized in this study and is discussed in the following section.

4.2.3. Calculating Rainfall Hyetographs Utilizing NEXRAD and GIS

Total storm rainfall amounts and rainfall hyetographs were calculated for each subwatershed utilizing NEXRAD and GIS data for the October 1994 storm event in the Clear Creek area. The resulting data was comparable to results that would have been obtained by 48 individual rain gages spread throughout the watershed, with one rain gage in each particular subwatershed. Rainfall hyetographs were developed at 30-minute intervals to coincide with the modeling time step used in the hydrologic model. The procedure for creating these storm totals and hyetographs follows.

As discussed above and in Chapter 3, radar data is both geospatially referenced and time-series formatted. Each individual radar data cell has a particular x,y-coordinate
with a corresponding grid value which indicates the estimated rainfall for that radar sweep. These properties allow gridded radar data cells to be summed over a particular area and time – thus representing the storm total rainfall for that area and time step of interest. A line shapefile schematic of the Clear Creek subwatersheds was layered with the radar data, providing the individual areas for cell summation and averaging. Figure 4.5 illustrates how the two data files can be combined to create a watershed wide storm total grid.

![Storm Total Rainfall Grid Shown with Outline of Clear Creek Subwatersheds](image)

**Figure 4.5**: Storm Total Rainfall Grid Shown with Outline of Clear Creek Subwatersheds

Individual hyetographs for each subwatershed were also created from the time-series radar data. On average, radar data was collected every 6 minutes during the storm event corresponding to the KHGX radar sweep time. The original rainfall data received
from Vieux and Associates was unformatted with rainfall estimates provided at each radar sweep. Despite variations in radar sweep times it was possible to bin this data into the desired 30-minute time step format. This process was done using Microsoft Excel and a Virtual Basic script. The resulting data was then ready for entry into a hydrologic model of the Clear Creek watershed. The resulting data for both the storm totals and individual rainfall hyetographs are discussed in Chapter 5.

4.2.4. Design Storm Hyetographs

A design storm is a theoretical precipitation event used as the basis of design for a hydrologic system. Key factors defining a design storm are the amount (volume) of

![Rainfall Isohyets (inches)]

Figure 4.6: TP-40 Rainfall Isohyets Used to Generate the 100-Year Recurrence Interval and 24-Hour Duration Design Storm (USDOC, 1963)
precipitation and its distribution, both temporally and spatially across a watershed.

Maps and data from Technical Paper No. 40 (TP-40) from the National Weather Service were used to provide the rainfall amount, frequency and duration data required for design storm development (USDOC, 1963). Figure 4.6 above shows an example of a TP-40 map illustrating the rainfall isohyets (lines of equal precipitation) for a 100-year, 24-hour storm event. These maps were obtained from Dodson and Associates and displayed within a GIS. The location of Clear Creek was readily discernible and a simple graphical interpolation between the nearest isohyets was performed to obtain the correct rainfall amounts for storms of varying duration and frequency. The resulting rainfall amounts are displayed in Table 4-2.

Table 4-2: Rainfall Amounts (inches) for Design Storm Generation

<table>
<thead>
<tr>
<th>Storm Frequency</th>
<th>Storm Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5 min</td>
</tr>
<tr>
<td>5-Year</td>
<td>0.9</td>
</tr>
<tr>
<td>10-Year</td>
<td>1.0</td>
</tr>
<tr>
<td>25-Year</td>
<td>1.2</td>
</tr>
<tr>
<td>100-Year</td>
<td>1.4</td>
</tr>
</tbody>
</table>

These design frequencies were selected for a variety of reasons. The 100-year design storm is the most common design storm frequency used by the National Flood Insurance Program (NFIP) to delineate at-risk properties. The 25-year design frequency was selected because it most closely matched the October 1994 historical storm with respect to intensity. The 10-year frequency was selected due to the 10-year design criteria of channel improvements planned by the Corps. The 5-year frequency was selected to help
identify houses and properties in the Clear Creek area that would be considered prime candidates for property buyouts.

It is important to note that the design frequency of a storm does not represent a time limitation for occurrence of an actual precipitation event but rather a conditional probability of that rainfall intensity occurring within a one-year period. For example, a design frequency of 100 years means that there is a 1% chance of a storm of that particular intensity occurring in any given year; similarly, a design frequency of 25-years equates to a 4% occurrence probability for any year.

The above listed rainfall intensities were entered into the HEC-HMS program to create the actual design storm hyetographs. The program uses the alternating block method to create hyetographs at the desired time-interval (USACE, 2000c). Another very important point to highlight is that the above listed rainfall intensities are point estimates, which need to be adjusted for use over larger watershed areas. In 1958, the U.S. Weather Bureau derived factors by which point precipitation values are to be reduced to yield areal-average intensities (USACE, 2000c). The reduction factors are functions of both the area of the watershed and the duration of the design storm. Specifics on the adjustment factors are available in the HEC-HMS Technical Manual. It is believed that NEXRAD radar data may soon be used to assist in more clearly defining this point-area relationship. The resulting design storm hyetographs used as input for the hydrologic model are shown in the following chapter.

4.3. Streamflow Data

All of the streamflow data utilized in this study was obtained from the U.S. Geological Survey (USGS), which has only one rated gage on Clear Creek that can
measure river flow rates. The Clear Creek Flow Gage (# 08077600) has been located at various positions along Clear Creek since it was originally placed into service in 1972. In 1994, it was located at the FM 2351 Clear Creek crossing (see Figure 4.1), and measured the direct runoff of approximately 90 square miles of the watershed. In 1996, the stream gage was moved approximately four river miles further downstream to its current location at the FM 518 Clear Creek crossing. The stream gage has a current drainage area of approximately 122 square miles.

Stream flow data was obtained by direct contact with the Houston office of the USGS. Data for the October 1994 storm event was obtained in spreadsheet format and utilized as an observed hydrograph for the purpose of calibrating the HEC-HMS hydrologic models, which is discussed in Chapter 5.

4.4. Improved Hydrologic and Hydraulic Modeling Programs: The U.S. Army Corps of Engineers' Next Generation Software

Computer estimations and representation of flooding are accomplished by using two different types of models: hydrologic and hydraulic. Hydrologic modeling refers to the determination of the hydrologic response of a basin to a rainfall event. This response is measured or computed in terms of flow rate. HEC-1 and HEC-HMS are two types of hydrologic models, developed by the Corps that are widely used throughout the country. Hydraulic modeling involves the estimation of the water surface profile, in terms of elevation, resulting from the flow rates calculated by the previously discussed hydrologic model. This profile, when combined with the pertinent ground surface elevation data (topographs), provides an estimate of the extent of flooding experienced during a time-period of interest. HEC-2 and HEC-RAS are the two most widely used hydraulic
programs in the nation (Maidment and Djokic, 2000). Both HEC-HMS and HEC-RAS are being continually updated as part of the Hydrologic Engineering Center’s “Next Generation” of engineering software intended for use with higher-end computers operating in a Windows environment (Hoggan, 1997). The following sections provide a brief discussion of the HEC-HMS and HEC-RAS modeling programs.

4.4.1. Hydrologic Modeling System (HEC-HMS)

The U.S. Army Corps of Engineers’ HEC-HMS is a Windows-based program with significant improvements over its predecessor, HEC-1. HEC-HMS (version 1.0) was released in March of 1998. Version 2.0, released in March of 2000, has eliminated many of the programming problems that plagued the first few releases. Several more versions will be released over the next 5 years. The Corps expects HEC-HMS to gradually replace the older, DOS-based HEC-1 within this same time frame, as fewer Corps studies will be initiated utilizing the HEC-1 format.

The Windows-based format allows for the program’s most notable improvement, a highly effective graphical user interface (GUI) illustrated in Figure 4.7. This interface permits easier input and manipulation of hydrologic elements that represent physical processes within the watershed such as a subbasin, a stream reach, or a confluence. The characteristics of each of these elements can be readily modified via pull-down windows. Results can also be more easily viewed at any point along a schematic of icons representing specific portions of the modeled watershed. The much improved graphics capabilities allow for the quick generation of easier-to-read hydrographs. An import feature permits the transfer of existing HEC-1 programs to the updated HEC-HMS format.
Another significant improvement of HMS is its method of organizing and storing data. The modeling data is organized into three separate sub-models. Version 2.0 of HMS refers to them as the basin model, the meteorological model, and the control model. Each sub-model is required for a specific overall model run.

The basin model contains pertinent information regarding the hydrologic system connectivity and other physical data describing the basin. It is the sub-model within which the watershed schematic is constructed and manipulated. Seven different hydrologic elements are used to construct a hydrologic model including: subbasin, reach, reservoir, junction, diversion, source, and sink. These elements are connected to the
schematic in such a way as to create a representation of the actual watershed. Each element requires specific input data for the mathematical equations it uses to describe the physical process. Loss rates, transform method and baseflow are also added or adjusted within the basin model (USACE, 2000c).

The meteorologic model contains the precipitation and evapotranspiration data necessary to simulate watershed processes. The precipitation data can be either historical or hypothetical. Historical data is rainfall data provided by either rain gages or NEXRAD for a past storm event. Frequency or design storms can be created and entered within the model (USACE, 2000c).

The control model contains the time parameters for execution of the model runs. The starting and ending dates and times of a run are entered in this sub-model. This sub-model also contains the specified computation step or time interval for the specific run.

This modularized method of entering, storing, and retrieving data greatly increases the flexibility of the program. Changes can be made to a particular sub-model without necessitating a change to the entire model. Additionally, a variety of different data sets may be entered under each sub-model and the user may then select the desired sub-models for a specific run without creating an entirely new overall model as was required in HEC-1. For example, minor adjustments to a basin schematic, such as those performed during the inclusion of a detention pond, can be made and included as a second basin model. Comparisons can be performed by selecting any two desired basin models for two corresponding computer runs. This flexibility makes modeling improvements or changes to a given watershed a far simpler process because HEC-1 required a separate data set describing all aspects of the modeling run for each independent execution of the program.
Another significant difference between HEC-HMS and HEC-1 is the use of the Data Storage System, or HEC-DSS, to manage time-series and tabular data. This system was created to make the management and transfer of similar hydrologic data between different programs more efficient (USACE, 2000e). The DSS employs a HEC-wide standardized method of data files that permits each of the programs to access, utilize and update the same database.

4.4.1.1. Development of the HEC-HMS Hydrologic Model for Clear Creek

Various sub-models (basin, meteorologic, and control) used in the HEC-HMS hydrologic model were created for the Clear Creek watershed. The baseline basin model was created by importing an existing HEC-1 formatted hydrologic model of the area, originally developed by the Dannenbaum Engineering Corporation (DEC). The meteorologic models were created by entering design storm criteria and by entering formatted NEXRAD radar data for the October 1994 historical storm. One control model was created to establish a 30-minute computational time step for all hydrologic calculations.

4.4.1.1.1. Creating the Basin Models

The import feature of the HEC-HMS software successfully reformatted an existing HEC-1 model of the Clear Creek watershed originally created by DEC for a Master Drainage Study conducted in the early 1990’s (DEC, 2000a). A thorough verification of basin parameter data found no discrepancies from the HEC-1 model.

The basin model consisted of 48 subwatersheds, 38 reaches and 7 overflow diversions. Each of the subwatersheds utilized the Clark unit hydrograph method for
subbasin transforms. Time of concentration (TC) and basin storage (R) values for each subwatershed were used as calibration parameters. Loss rates were calculated using the Initial / Constant method in accordance with the Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas created by the Harris County Flood Control District (HCFCD, 1984). Baseflow was not included in any of the hydrologic model runs. A total of 33 of the 38 reaches utilized the Modified-Puls method for routing calculations. The remaining five reaches used the Lag method and were associated with 5 of the 7 overflow diversions. The 7 overflow diversions routed excessive flows from one region of the watershed to others. Although included in the hydrologic model, no further discussion of the overflows will be included in this report as their overall impact to observed flows was minimal.

4.4.1.1.2. Creating a Map File for the Basin Model

One of the most useful features of the HEC-HMS software is the ability to create a map file of the modeled watershed. The map file consists of a series of X,Y coordinate pairs delineating the subwatershed boundaries and river reaches in an orthographically correct fashion. The resulting basin and river reach map, although playing no role in the hydrologic computations, is extremely useful in geographically organizing the individual subwatersheds and river reach model icons. The map file image can be seen in Figure 5.9, combined with the basin model schematic.

Creating the map file of the Clear Creek watershed was accomplished by creating ASCII files from existing GIS shapefiles of the Clear Creek subwatersheds and river reaches. These shapefiles were the same used to produce the schematic representations seen in Figure 4.2. It is important to note that the map file must be created prior to
development or import of the basin schematic. The current version of HEC-HMS does not provide a map scaling option and map files entered into the program after schematic creation will be excessively minute in scale compared to the schematic (USACE, 2000c).

4.4.1.1.3. Creating the Meteorologic Models

A total of six meteorologic models were created, representing both various design storms and the historical October 1994 rainfall event. The historical meteorologic models utilized NEXRAD radar data for reasons discussed earlier in this report.

Design storms were created utilizing the HEC-HMS software and the rainfall amounts, storm duration and frequency data shown in Table 4-2. The approximate storm area (in square miles) must also be entered. Design storm calculations were previously discussed in this report. The resulting 24-hour hyetographs for each design storm are discussed in Chapter 5. These 24-hour hyetographs were applied to each of the 48 subwatersheds in the model.

Historical storm creation utilizing NEXRAD data as input required significantly more effort than that involved with design storm generation. This is due to the fact that unlike design storms, which are considered uniform over the watershed, historical storms vary in intensity across the watershed. As discussed previously, the improved spatial resolution of the data provided by NEXRAD allowed the creation of specific rainfall hyetographs for each subwatershed. This variation in the rainfall pattern from subwatershed to subwatershed necessitated the creation of 48 different precipitation gages within the meteorologic model. Sample October 1994 rainfall hyetographs utilized as input to the model can be seen in Chapter 5.
One control model was created for the overall hydrologic model to establish the start and stop dates and times of the model run as well as the time interval for computations. Start and stop dates were entered to correspond to the desired start and stop dates of the watershed outfall hydrograph resulting from any rainfall event. This corresponded to a start date/time of 0000, 14OCT94 and a stop date/time of 2400, 24OCT94. A 30-minute time step established for hydrologic computations.

4.4.2. River Analysis System (HEC-RAS)

The Corps' River Analysis System (RAS) is a Windows-based program that contains an integrated package of hydraulic analysis programs designed to replace its predecessor, HEC-2. HEC-RAS is nearly identical to HEC-2 computationally, with only a few minor changes. The first version of HEC-RAS was released in 1994 and is therefore currently experiencing more widespread use than its next generation counterpart HEC-HMS. The current version of HEC-RAS, version 2.2 released in September of 1998, is the preeminent one-dimensional steady-flow water surface profile program in use throughout the United States (Maidment and Djokic, 2000). Future versions of HEC-RAS will include unsteady flow and sediment transport analysis features. In fact, the beta-version of HEC-RAS v3.0, which includes unsteady flow analysis, has been released on a limited basis for testing.

HEC-RAS is comprised of a graphical user interface (GUI), a separate hydraulic analysis component, data management and storage capabilities, graphics tools, and reporting utilities with each presenting significant improvements over the DOS-based HEC-2 program. The GUI is again, one of the most significant advantages of the program. Operationally similar to the HEC-HMS interface, the GUI operates off a series
of pop-up windows and pull-down menus (see Figure 4.8). These provide a variety of functions including file management, data entry and editing, execution of hydraulic simulation, display of input and output data, printing of plots and tables, as well as online help. The powerful data management and storage capabilities are attributable to the integration of the DSS discussed previously. The reporting utility provides a wide-array of options for generating custom reports that allows the user to focus on particular portions of the vast amounts of data generated by a hydraulic analysis (USACE, 2000e).

Figure 4.8: HEC-RAS Graphical User Interface Showing Sample Data in the Geometric Editor with Inset Pop-Up Window Showing Cross-Sectional Elevation Data (USACE, 2000d)
Additional information about these features of the HEC-RAS suite is available from the HEC-RAS User’s Manual (http://www.hec.usace.army.mil). The hydraulic analysis component and graphics tools available in this suite will be discussed here in slightly greater detail.

Hydraulic analysis is accomplished through the GUI on the basis of “projects.” Each project identified in the program is associated with a particular river system. Each project consists of a series of “plans.” A plan is divided into two categories of data: geometric data and flow data. Within each project, various plans can be developed from different combinations of geometric and flow data, together with boundary conditions and run specifications. The geometric data and flow data is handled via their respective editors.

The geometric data editor is where all physical and topographical data is input and displayed during model creation. Required data entry includes the river network including river reaches from upstream to downstream and cross-section elevation data. Cross-sectional data is entered in its own editor and each cross-section is identified by reach name and river station. Each cross-section can contain a maximum of 500 station-elevation coordinate points. This data must be entered manually and is usually obtained from field surveys; however, digital elevation data may also be used. Additional data is entered in this editor such as distances to the next downstream cross-section for the channel centerline and left and right overbanks. Also, Manning’s n-values are entered for each cross-section. Lastly, hydraulic features may be entered using similar data editors to the cross-section editor. Hydraulic features that can be modeled by HEC-RAS include: bridges, culverts, weirs, and spillways (USACE, 2000e). Each of these features requires
substantial additional data to accurately determine their hydraulic influence. A detailed
discussion of each of these is far beyond the scope of this report.

Flow data is entered via its own editor. For steady-flow data, it is necessary to
specify the number of flow-elevation profiles to be computed, the discharge or flow
values corresponding to each profile, and the boundary conditions. Flow data is usually
obtained from a previously created and executed hydrologic model such as HEC-HMS.
HEC-RAS requires that the user select the reach and all the cross-sections where a
change in flow occurs according to the hydrologic model. This data entry feature can
either be done manually or may be performed via the DSS. Once a specific flow is
entered for a given cross-section, the program utilizes that flow value for all downstream
cross-sections until a cross-section with a different flow is encountered. HEC-RAS also
permits the ability to model and display multiple flow profiles simultaneously. For
example, a variety of different flow values may be entered for a given cross-section with
each corresponding to a different design storm flow. Therefore it is possible to easily
compare the elevation profiles for the 5, 10, 25, and 100-year design floods. The
required boundary condition data for each desired profile is dependent on the type of flow
regime. A flow regime may be supercritical, subcritical or mixed and the corresponding
boundary condition must be at the upstream, downstream, or both ends of the river reach
respectively (USACE, 2000f). It should be noted that for most floodplain analyses in the
Houston area, subcritical flow is expected and boundary conditions should therefore be
established at the downstream end of a modeled river reach (HCFCD, 1984).
4.4.2.1. Development of the HEC-RAS Hydraulic Model for Clear Creek

The following is a brief summary of the overall methodology used in developing the Clear Creek hydraulic model. First, a new project must be entered and appropriately titled. Second, geometric data files must be created including data such as: river reach lengths, cross-sectional station and elevation data, as well as hydraulic structures and other required information. Third, flow files must be created that include the flow and boundary data for each desired flood profile to be analyzed. Fourth, a plan must be established by selecting a specific geometric data file and flow data file from those created in the previous two steps. Finally, the model may be executed. Evaluation of the model may be performed by careful analysis of a variety of display options available in the program.

4.4.2.1.1. Creating Geometric Data Files to Model the Effects of Channel Modifications

A total of six different geometric data files were created to represent a variety of hydraulic conditions being investigated. These conditions ranged from the baseline geometric conditions of the creek as it exists presently to a variety of altered conditions representing proposed channel modifications for flood control. The geometric data files are presented as various structural flood control scenarios and are summarized below in Table 4-3.

The various scenarios represent attempts to model the effects of a limited channelization scheme through the Friendswood, Texas area. The baseline geometric file was created from a HEC-GeoRAS input file derived from digital GIS data of the Friendswood area. The development of the HEC-GeoRAS input files is discussed in Section 4.5.2.1.
Table 4-3: Summary of Geometric Data Files for the Hydraulic Study Region

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Condition</th>
<th>Resulting Reach Length / Percentage of Reach Modified</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Baseline / Existing (NC)</td>
<td>6.5 miles / 0.0%</td>
</tr>
<tr>
<td>2</td>
<td>Long, Widened Channel with 50-meter base width. No straightening. (LW50)</td>
<td>6.5 miles / 0.0%</td>
</tr>
<tr>
<td>3</td>
<td>Short, Widened Channel with 30-meter base width. Channel straightened. (SC30)</td>
<td>4.9 miles / 64%</td>
</tr>
<tr>
<td>4</td>
<td>Short, Widened Channel with 10-meter base width. Channel straightened. (SC10)</td>
<td>4.9 miles / 64%</td>
</tr>
<tr>
<td>5</td>
<td>Long, Widened Channel with 30-meter base width. Channel straightened. (LC30)</td>
<td>4.4 miles / 84%</td>
</tr>
<tr>
<td>6</td>
<td>Long, Widened Channel with 10-meter base width. Channel straightened. (LC10)</td>
<td>4.4 miles / 84%</td>
</tr>
</tbody>
</table>

The baseline file was used to model the current, existing topography and channel morphology through the reach of interest that runs for approximately 6.5 miles. It should be noted that two major hydraulic structures (bridges) are present in the reach and that the modeling data for these structures was taken from an existing HEC-2 model created by DEC (DEC, 2000b).

The remaining scenarios were created to model the effects of various channel modifications to the existing channel morphology. As discussed in Chapter 3, channel modifications improve the hydraulic efficiency of a natural channel through a variety of mechanisms including increasing the channel width, depth and slope, as well as channel straightening and smoothing. As there are clearly a large number of possible modeling combinations, only a select few were used for modeling in this thesis.
Altered channel depth was increased equally in scenarios 2 through 7 to reduce the lateral extent of the channel improvements and thereby reduce the required right-of-way for channel construction and maintenance. Channel slope was maintained approximately equal to the existing channel slope at .0003 meter/meter. The channel modification was a constant, trapezoidal shape through the entire modified reach. Channel side slopes were always 3:1 (horizontal : vertical). All channel scenarios would include a grass-lined channel only with no concrete reinforcement or lining.

Four factors were varied in scenarios 2 through 7 to obtain a relative indication of the effectiveness of specific channel designs. These factors included channel: width, straightening, length and smoothing. Channel width was altered using a feature in HEC-RAS called Channel Improvement (CHIMP). The CHIMP feature automatically adjusts cross-sectional topographic data to user-specified channel widths, side slopes, bed and bed slope. Channel smoothing was modeled by altering the Manning's roughness coefficient to account for changes in the friction losses associated with water flowing over the bottom and sides of the channel. Channel straightening and length were modeled by changing the HEC-GeoRAS input files to reflect changes in the plan view of the existing reach.

Scenario 2 models the effects of a 50-meter wide channel that follows the meandering path of the existing channel. Scenarios 3 and 4 model the effects of a short, straightened channel for two different channel bottom widths of 10 meters and 30 meters respectively. Scenarios 5 and 6 model the effects of a longer, straightened channel for the same two bottom widths of 10 and 30 meters respectively. Both the short and long straightened channels reduce the effective reach length of the original hydraulic study area. The shorter channels modify the morphology of approximately the upper 4.15
miles (or approximately 64%) of the natural reach length. The longer channels modify approximately the upper 5.45 miles (84%) of the natural reach length. Results of the various channel improvements and further justification for their selection are provided in Chapter 5.

4.4.2.1.2. Creating Flow Data Files for the HEC-RAS Model

Two different flow files were created for use in the HEC-RAS model with one corresponding to the flows created by the design storms and the other to flows generated by the historical Oct 1994 storm. Both flow models used data provided by the HEC-HMS model previously discussed in this chapter, with flow changes occurring at specified stations along the hydraulic study area. A table of flow change locations and the specific flows computed for each design storm as well as the historical storm is presented in Chapter 5.

As discussed previously, the flow models also require boundary conditions at which the water surface elevation calculations are commenced. All flows within the Clear Creek watershed remain within the subcritical region; therefore, all boundary conditions must be dictated at the furthest downstream point of the study reach. Known water surface elevations at the downstream end of the hydraulic study reach were obtained from the existing HEC-1 hydraulic model (DEC, 2000a) and used as boundary conditions for the various design storm profiles.

4.5. Geographic Information Systems

Since almost all hydrologic data has some geographical component, a technology such as GIS allows for the analyzing of the spatial component of this data to possibly
reveal trends, relationships and patterns. A GIS allows for the integration of this
graphical data, often in the form of maps, with descriptive or attribute information from a
wide-variety of existing databases. Therefore, a GIS becomes much more than a
mapping tool, as it is often times considered. It is a database that can be queried and
manipulated as any other database management tool but one with the additional
advantage of displaying data in a more understandable format. Another significant
advantage of this type of data management and display is that critical relationships
between these varying data are often only recognizable when displayed in a geo-
referenced framework.

Throughout the 1980’s and 1990’s, advances in computer science and
microprocessor technology have allowed for the development of a wide-array of
powerful geographic information system software. Commercially available GIS
packages such as ERDAS and ARC/INFO are capable of handling both raster and vector
based data entry, editing, display, analysis, modeling and management. Some, such as
ERDAS are more suited to raster image processing while others, such as ARC/INFO, are
more fully functional. ARC/INFO, developed by the Environmental Resource Institute
(ESRI, Inc.) is one of the most widely used packages throughout the world (Mitchell, 1999). However, many of these powerful programs require significant financial
investments and training to realize their full potential. As a result, more cost-effective
and easier to utilize versions of these programs are becoming available.

4.5.1. ArcView

ArcView is a cost-effective and easy-to-use GIS software package developed by
ESRI that brings geographic information management, analysis and mapping to the
personal computer. The program enables users to integrate vector and raster data, photographs and satellite images, scanned documents, CAD drawings and a wide range of other data types for display, query, and analysis. A variety of available plug-ins, known as extensions, permit additional functions. Many of these extensions are custom designed for specific applications in certain fields such as hydrology and floodplain mapping.

While not as powerful as its more functional counterpart, ARC/INFO, ArcView is significantly more user-friendly and is more affordable. Combined with a few specifically designed extensions, ArcView can approach the functionality of ARC/INFO in a particular field of choice without wasted investment. Some of the particular ArcView extensions used in this study are discussed below.

4.5.1.1. Spatial Analyst Extension

The ArcView Spatial Analyst extension provides the user a broad range of powerful spatial modeling and analysis features. This extension permits the creation, display, and querying of cell-based raster data. This feature is vital for the use and display of continuous surface modeling information such as gridded rainfall and digital elevation models, which cannot be modeled as vector data. Spatial Analyst also performs integrated raster-vector theme analysis. This allows for the aggregation of properties in a raster theme based on an overlaid vector theme. An example of the use of this feature is the creation of individual subwatershed hyetographs by combining rasterized rainfall data from NEXRAD and a vector based polygon theme representing the borders of the subwatershed as discussed in Section 4.2.3.
These features, as well as others, have not previously been available to desktop users. A brief summary of additional features provided by Spatial Analyst is provided below:

- Convert feature themes (point, line, or polygon) to grid themes,
- Create continuous surfaces from scattered point features,
- Derive contour, slope, and aspect maps of these types of surfaces,
- Perform cell-based map analysis such as map algebra,
- Import data from standard formats such as the USGS DEMs.

4.5.1.2. 3-D Analyst Extension

The ArcView 3-D Analyst extension provides advanced tools for three-dimensional modeling and analysis. It enables users to create, analyze and display surface data with support for triangulated irregular networks (TINs) and simple three-dimensional vector geometry. A TIN is vital for the formation of computer-generated floodplains by providing the topographic base map in 3-dimensional form; thus, allowing the water surface profiles calculated by HEC-RAS to be combined with the terrain model and therefore determine the extent of the floodplain. Examples of this type of data are provided in Chapter 5.

3-D Analyst also provides several other additional features including the ability to:

- Generate three-dimensional contours,
- Integrate data from computer-aided design (CAD),
- Build true 3-D surface models from any point data source such as GPS,
• Drape two-dimensional features or image data on three-dimensional surfaces and have complete access to tabular data via interactive query.

3-D Analyst also provides for a variety of interactive viewing options with the ability to change perspectives, zoom, pan, create fly-by video files and others.

4.5.2. Hec-GeoRas Extension

The HEC-GeoRAS extension is a specifically designed set of procedures, tools and utilities for the processing of geospatial data for use with HEC-RAS, linking the data development and display capabilities of a GIS with a powerful hydraulic modeling program. The extension allows users to create an import file containing geometric attribute data from an existing digital terrain model (DTM) and selected complementary data sets such as river reaches, right and left overbanks and others. This file will be referred to as the HEC-RAS GIS import file. Post-hydraulic analysis results generated by HEC-RAS can also be exported back to HEC-GeoRAS and converted to a GIS format for spatial analysis and floodplain mapping. This file is referred to as the HEC-RAS GIS export file. Both Spatial Analyst and 3-D Analyst are required to achieve the full functionality of the Geo-RAS extension.

The current version of HEC-GeoRAS creates a HEC-RAS GIS import file containing user-defined river, reach and station identifiers; cross-sectional topographic elevation lines; cross-sectional bank stations; downstream reach lengths (between cross-sections) for the left overbank, main channel, and right overbank; and cross-sectional roughness coefficients (USACE, 2000h). HEC-RAS utilizes this data to develop geospatially correct hydraulic models of the system. Hydraulic structures such as
bridges, culverts, etc., are not included in the import file created by GeoRAS and must be
entered directly to the model within HEC-RAS. After the addition of the required flow
data, the HEC-RAS program performs its regular calculations and generates the desired
output data for each flow profile entered as discussed in section 4.4.2. However, the
water surface profile data and velocity data may now be exported from HEC-RAS to
HEC-GeoRAS, where it is processed into GIS data layers.

These layers include water depth grids, velocity grids, and floodplain polygons,
which when combined with base maps such as Digital Orthophoto Quarter-Quadrangles
(DOQQs), serve as excellent digitized floodplain boundary maps. The GIS format of this
output data can be put to a variety of uses including: inundation comparisons between
different design storm events, calibration of the hydraulic model by comparing the
floodplain with structures known to be flooded during a specific historical event, and
determining the effectiveness of a variety of different flood control methods. Figure 4.9
will assist the reader in understanding the flow and exchange of information between the
HEC-GeoRas extension and the HEC-RAS model.

4.5.2.1. Creating the HEC-RAS GIS Input File

A HEC-RAS GIS input file was created for the area surrounding the hydraulic
study reach. The input file consisted of a series of shapefiles representing the stream
centerline, stream banks, overbank flowpath centerlines, cross-sectional cut lines and a
newly created land use theme. Additionally, the input file contained a Digital Terrain
Model (DTM) to represent the surrounding topography and provide baseline elevation
data for hydraulic analysis.
Figure 4.9: Flowchart of Methodology and Data Transfer Between HEC-RAS and HEC-GeoRas (USACE, 2000h)
The DTM was in a Triangulated Irregular Network (TIN) format and was created using ArcView's 3-D Analyst from a 10-meter resolution Digital Elevation Model (DEM). The 10-meter resolution DEM was chosen because all cross-sectional elevation data used in the HEC-RAS model would be extracted from the intersection of these transect lines and the DTM.

The required shapefiles were manually created in ArcView using a 1-meter Digital Orthophoto Quarter Quadrangle (DOQQ). The 1-meter DOQQ was found to provide sufficient resolution to trace stream centerlines, approximate channel banks, and estimates of overbank flowpath centerlines. Although an unlimited number of cross-sectional elevation transects are permitted, the cross-sections were patterned after and limited to those used in the original HEC-2 analysis conducted by DEC for comparison purposes. These shapefiles were edited as required to represent channel modifications made to the stream. Illustrations of the HEC-RAS GIS input file data can be seen in Figures 4.10 (DEM), 4.11 (TIN), and 4.12 (DOQQ with input shapefiles).

4.6. Modeling Floodplain Property Buyouts

As discussed in Chapter 3 of this report, one of the significant reasons behind the past non-implementation of non-structural flood control measures, such as floodplain property buyouts, was the inability to directly model the costs and benefits associated with these types of alternatives. This research developed an indirect modeling method that permits rapid and accurate identification of "at-risk" properties and the dollar costs
associated with their relocation. The approach applies the above discussed floodplain generation methodology with GIS mapping of properties within and near the floodplain and is discussed in brief detail in this section.

All property locations in the Friendswood area within the Clear Creek 100-year floodplain as indicated by the Federal Emergency Management Agency's (FEMA) Flood Insurance Rate Maps (FIRMs) were identified (FEMA, 1996a, 1998, and 2000). This was accomplished by overlaying the FEMA 100-year floodplain on property plat maps obtained from the Appraisal Districts of both Harris and Galveston County. Once the addresses were obtained, it was possible to obtain property values for each of the sites. Additionally, the properties were address geocoded in ArcView, resulting in a digital representation of the location of the 100-year floodplain properties.
Address geocoding is a process in which a list of addresses is compared to a reference theme in GIS and matched to their respective geographical locations on the reference theme. The reference theme is usually a high-quality digital street or road map file with attributes that specify the street name, street type and the range of addresses that occur along each street. It should be noted that geocoding is a matching process with the possibility of significant error attributable to a variety of factors. Some of these include address matching errors and possible errors in the street reference theme. In an attempt to minimize some of this error, the 1-meter DOQQs were used as a quality check of the house locations by ensuring that properties existed where the digital geocoding process
placed the digital representation of that address. Limited on-site verifications were conducted to verify specific streets of interest with a large number of properties and where the DOQQ did not provide sufficient proof of an existing property. Although it is not readily quantifiable, the reader should assume a 10-meter error with each of the digital representation of property locations. The result of the above efforts was the creation of a digital, georeferenced database of the addresses, location, and property values of all the FEMA identified 100-year floodplain properties along Clear Creek in the Friendswood area. Appraisal district data is included in Appendix A and Appendix B of this thesis. Street numbers were omitted from the data sheets for Privacy Act concerns.
Once the digital representation of the 100-year FEMA flood properties was available in ArcView, it was possible to compare their locations with our calculated design storm and historical storm floodplains. These floodplains were created by importing the generated HEC-RAS GIS output files into HEC-GeoRas. HEC-GeoRas then creates water surface elevation TINs that correspond to the HEC-RAS calculated water surface elevations. The program then combines these water surface elevation TINs with the original DTM. The intersection of these two TINs demarcates the extent of the floodplain. Shapefiles are then created inside of the GIS program that correspond to these areas. Queries were conducted in the ArcView database to determine an estimate of the dollar values associated with purchasing properties within the various design storm floodplains. A more complete discussion of the calculated dollar amounts associated with purchasing properties within the floodplain is included in Chapter 5.
Chapter 5. Results and Discussion

This chapter will present results in the order of the objectives listed in Chapter 1 of this report. As discussed in Chapter 4, the sequence of these objectives is of vital importance to the overall goal of performing floodplain analyses. Calibration results of both the hydrologic and hydraulic models are included in this section.

5.1. Rainfall Analysis using NEXRAD Radar

The use of NEXRAD radar to estimate rainfall amounts was necessitated by the sparse location of rain gages throughout the Clear Creek watershed. Following the methodology discussed earlier, individual subwatershed hyetographs were generated for input into the HEC-HMS model of the Clear Creek watershed. These hyetographs represented both the amount and timing distribution of the rainfall over the entire watershed. Figure 5.1 shows the subwatersheds in the Clear Creek hydrologic study area.

Figure 5.1: Subwatersheds Within the Clear Creek Hydrologic Study Area
with their corresponding labels. This figure will be referenced periodically throughout
the remainder of this section to aid the reader in locating specific areas of interest
throughout the watershed.

5.1.1. Total Storm Rainfall Amounts by Subwatershed

Spatial Analyst, an ArcView extension discussed in Chapter 4, was used to create
Figure 5.2. This figure shows a raster (cell-based) data display of the rainfall totals over
the Clear Creek watershed. Each individual cell in this figure represents the total
estimated rainfall in inches over that particular area. The reader should notice the
variations in cell areas ranging from the smallest 1 km² grid cells to the largest 4 km² grid
cells. This variation was caused by projection changes from the original radar reflectivity
projection (HRAP) to the Universal Transverse Mercator (UTM) Zone 15 projection. A
discussion of projections is beyond the scope of this report. However, more information
on this topic can be found in any standard GIS text such as Distributed Hydrologic
Modeling using GIS by Dr. Baxter E. Vieux. Figure 5.2 was created by summing each of
the individual radar sweeps over the time period between 3:30 PM on October 14th, 1994
and 10:30 AM on October 19th, 1994. Figure 5.3 shows the NEXRAD data storm totals
for each of the subwatersheds. These subwatershed totals were achieved by averaging
each of the respective radar grid cell totals within the subwatershed boundaries. Areally
weighted averages were used to account for data cells that intersected subbasin
boundaries and as a result, contributed rainfall to two or more subwatersheds.

A brief summary of the procedure used by Vieux and Associates to calculate the
radar rainfall totals illustrated in both figures is shown below.
Figure 5.2: Storm Total Rainfall Grid Shown with Outline of Clear Creek Subwatersheds

Figure 5.3: Storm Total Rainfall on a Subwatershed Basis
1) Reflectivity data is extracted for the desired location and time period from archived NEXRAD data.

2) A rainfall rate is calculated for each individual radar cell and time period by using the tropical Z-R relationship discussed in Chapter 3. The relationship is as follows:

\[ Z = 250 \, R^{1.2} \]

Where: \( Z = \) reflectivity

\( R = \) rainfall rate in mm/hr

3) The rainfall rates are converted from mm/hr to in/hr.

4) Rainfall depth for each cell is calculated by the following equation:

\[ \text{Depth} = \frac{((\text{Rate}(i) + \text{Rate}(i-1))/2)}{\text{timestep}} \]

This equation takes the rainfall rate at the end and beginning of each timestep and calculates an average rainfall rate for that timestep. Multiplying this average rainfall rate by the timestep results in the depth of rainfall over that time period. The timestep varies slightly but is usually just under six minutes. Summing each calculated depth for each cell over the entire storm duration results in data such as that shown in Figure 5.2.

5) Spatial Analyst and ArcView are then used to calculate the average rainfall depth for each timestep over each subbasin by using a process called map algebra. This average rainfall amount is calculated using the following equation:

\[ \text{Average Rainfall (inches)} = \frac{\Sigma (\text{Depth}(i) \times A(i))}{\Sigma (A(i))} \]
Where:  \( \text{Depth (i)} = \text{Rainfall depth for each grid cell in that subbasin} \)
\( A \ (i) = \text{The area of each grid cell in that subbasin} \)

A careful comparison of Figures 5.2 and 5.3 will show that ground clutter (areas of questionably high reflectivity) presented a problem in four of the 48 subwatersheds: A100A1, A100D, HI100B, and A111A. Ground clutter was identified as any radar rainfall cell that had a total storm grid amount greater than twice the surrounding grid averages. Replacing the affected radar grid cells with an average data value calculated from the surrounding unaffected cells effectively “masked” the ground clutter. This masking had a minimal effect on the storm rainfall totals for both A100D and HI100B. However, both A100A1 and A111A had significantly reduced post-masking rainfall storm totals. A100A1 was reduced from a total storm amount of 31.88 inches to a more reasonable 16 inches. A111A was reduced from 27.73 inches to 10.52 inches. The ground clutter observed in A111A can be attributed to Interstate 45 and its surrounding development. The clutter in subbasin A100A1 could not be correlated to either natural or man-made interference.

Figures 5.2 and 5.3 also illustrate that the western and northwestern portions of the watershed received significantly greater rainfall. The average watershed wide rainfall for the storm event was 13.5 inches. An Audio Video Interleaved (AVI) file created of the radar data revealed that the associated storm fronts were moving in a northwesterly direction. Additionally, significantly higher intensity storm cells can be noted in the northwestern portion of the watershed throughout the AVI. However, the storm had a few major squalls that moved primarily in an easterly direction. Figures 5.4 and 5.5 show
two snapshots from the radar AVI file showing the progression of one of these major squalls over the watershed.

In addition to summing the radar rainfall data over the entire storm duration, it was necessary to bin the original 6-minute interval data into 30-minute timesteps for input into the hydrologic model. This step is necessary in any hydrologic study to account for the timing distribution of rainfall throughout the watershed that could have dramatic impacts on the outfall hydrographs. The methodology for this procedure was discussed in Chapter 4. Figure 5.6 illustrates a sampling of two of the calculated subbasin hyetographs, MA100A and CW100D. These two subbasins were chosen to highlight the differences in rainfall patterns between the western and eastern sides of the watershed. The hyetographs illustrate a two-day period from the overall storm duration of five days, during which over 70% of the total storm rainfall occurred.

Figure 5.6 highlights the fact that the total amount of rainfall over a given area is a function of both intensity and duration. Although subbasin CW100D experienced two half-hour periods of intense rainfall greater than any 30-minute periods in subbasin MA100A, the total rainfall in CW100D was still significantly less. The rainfall depths for MA100A and CW100D over the two-day time period in the figure were 13.8 inches and 9.1 inches respectively. Additionally, the peak intensities for each of the subwatersheds correspond to the direction and speed of the associated weather pattern. The highest intensity peak in Figure 5.6 corresponds to the same time period illustrated in the radar AVI seen in Figure 5.5. The 30-minute to 1-hour delay in peak rainfall intensity experienced by these two subwatersheds is clearly seen in both the AVI figures and the hyetographs.
Figure 5.4: NEXRAD Rainfall Intensity Image for October 18\textsuperscript{th}, 1994 at 3:00 A.M. (CST)

Figure 5.5: NEXRAD Rainfall Intensity Image for October 18\textsuperscript{th}, 1994 at 4:00 A.M. (CST)
5.1.2. Design Storm Hyetographs

Figure 5.7 shows the 24-hour design storm hyetographs used as data entry into the hydrologic model. The methodology for creation of these design rainfall patterns was discussed in Chapter 4. The figure shows the design storms after areal adjustments were applied. Table 5-1 shows the 24-hour rainfall totals both prior to and after the adjustment along with the percentage adjustment applied by HMS.

As discussed in Chapter 4, the areal adjustments applied to the point rainfall amounts by HMS were to account for the expected reductions in the rainfall amounts when these point estimates are applied to areas larger than 10 square miles. It was expected that the percent adjustment would remain the same for each of the storm
Figure 5.7: 24-Hour Design Storm Hyetographs for Clear Creek

Table 5-1: Percent Adjustment to Point Rainfall Amounts for Use in the 24-Hour Duration Design Storms

<table>
<thead>
<tr>
<th>Storm Frequency</th>
<th>24-Hour Design Storm Total Rainfall Amounts</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Point Rainfall Amounts</td>
<td>Areally Adjusted Amounts (122 sq mi)</td>
</tr>
<tr>
<td>5-Year</td>
<td>7.1</td>
<td>5.8</td>
</tr>
<tr>
<td>10-year</td>
<td>8.5</td>
<td>7.8</td>
</tr>
<tr>
<td>25-Year</td>
<td>10.0</td>
<td>9.0</td>
</tr>
<tr>
<td>100-Year</td>
<td>13.0</td>
<td>12.0</td>
</tr>
</tbody>
</table>

frequencies, as the adjustment factors recommended by the National Weather Service (USACE, 2000c) are solely functions of watershed area (122 square miles) and design
storm duration (24-hours). The expected percent adjustment for each of the four storm frequencies was 92%. The variation in the adjustment applied by HMS could not be explained and should be researched further. For consistency between the various HEC models, the design storms utilized in the hydrologic model were adjusted as shown in Table 5-1.

5.2. Hydrologic Modeling Results

A HEC-HMS model of the Clear Creek watershed was created from an existing HEC-1 model with acceptable results. The specifics behind the actual HEC-HMS program were discussed in Chapter 4. This section presents the two basin schematics used in modeling the watershed. The first basin schematic represents the entire Clear Creek watershed and was used in developing hydrographs (streamflow versus time) for areas of interest throughout the watershed in response to the design storm inputs discussed in the previous section. The second basin schematic, used to model the October 1994 historical event, represents the lower portions of the watershed downstream of the single streamflow gage located on the creek. The streamflow gage observed data was used as input to the model to increase the accuracy of the ensuing hydraulic model of the area.

5.2.1. Watershed-Wide Basin Schematic and Map File

Figure 5.8 shows an HMS schematic representation of the basin-wide model developed by importing an existing HEC-1 file of the watershed. The figure also illustrates the basin boundary and river map files created by using ArcView. Each of the
four design storms were entered into the model as separate meteorologic models. Separate runs were completed with each of these meteorologic models. Results of the model runs included hydrographs of streamflow versus time for each of the subwatersheds, reaches, diversions and junctions shown in Figure 5.8.

It is important to emphasize the large amount of data generated by each of these runs. Recalling that hydrologic data is time-series in nature, it can be understood why a separate Data Storage System (DSS) was developed by the Corps to handle these large amounts of data. For simplicity, resulting flow rates will be presented for only four junctions located within the hydraulic study region.

5.2.2. Calibration of the Hydrologic Model

From a hydrologic standpoint, the October 1994 storm event in the Clear Creek watershed presented an excellent opportunity for hydrologic model calibration. Rainfall
amounts were close to or in excess of the 25-year design storm (24-hour duration) for 4 of the 48 subwatersheds including MA100A, MA100B, HI100A and A100B6. Table 5-2 shows the rainfall amounts experienced by each of these subwatersheds over their most intense 24-hour period of rainfall (0000, 18Oct94 to 0000, 19Oct94) compared to the 25-year design storm rainfall amounts adjusted for the area of each subwatershed. The majority of the remaining watersheds experienced rainfall amounts around the 10-year design storm criteria.

Table 5-2: Comparison of Select Watersheds to the 25-Year Design Storm Rainfall Amount

<table>
<thead>
<tr>
<th>Subwatershed</th>
<th>Rainfall Total Over Most Intense 24-Hour Period (inches)</th>
<th>Areaally Adjusted 25-Year Design Storm Amount (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MA100A</td>
<td>10.4</td>
<td>10.0</td>
</tr>
<tr>
<td>MA100B</td>
<td>9.4</td>
<td>10.0</td>
</tr>
<tr>
<td>HI100A</td>
<td>9.9</td>
<td>10.0</td>
</tr>
<tr>
<td>A100B6</td>
<td>9.7</td>
<td>10.0</td>
</tr>
</tbody>
</table>

Unfortunately, the location of the USGS streamflow gage at the FM 2351 Clear Creek bridge crossing limited the gaged area of the watershed to only 55% of the total Clear Creek watershed area. Therefore, only 55% of the basin-wide model could be calibrated using this gage. Nevertheless, calibration runs were made using the October 1994 event as an observed event.

Figure 5.9 shows the calibration hydrograph showing both the observed and modeled flow rates over an 11-day period ranging from 14 October 1994 through 24 October 1994. Table 5-3 summarizes the comparison between the observed and modeled hydrographs by comparing their peak outflows, date and time of peak, and the total
discharge. The total discharge of a hydrograph is measured in inches and is calculated by determining the area under the curve (flow rate times time) and multiplying the result by the area of the gaged watershed with resulting units of length.

Table 5-3: Comparison of the Observed and Modeled Hydrographs at USGS Stream Gage #08077600 Located Near FM 2351

<table>
<thead>
<tr>
<th>October 1994 Calibration Run</th>
<th>Peak Outflow (cfs)</th>
<th>Date / Time of Peak</th>
<th>Total Discharge (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed</td>
<td>7518</td>
<td>18Oct94, 2130</td>
<td>14.1</td>
</tr>
<tr>
<td>Modeled</td>
<td>7712</td>
<td>18Oct94, 1930</td>
<td>11.5</td>
</tr>
<tr>
<td>Difference</td>
<td>+ 2.6%</td>
<td>2 hours early</td>
<td>- 18.4%</td>
</tr>
</tbody>
</table>
The modeled results were in excellent agreement with the observed results with respect to peak outflow and time of peak. However, an 18.4% disagreement between the observed and modeled total discharge was considered unacceptable.

Attempts at optimizing the subbasin parameters were conducted using the HEC-HMS parameter optimization feature. Parameters such as initial loss rate, constant loss rate, time of concentration and storage coefficients were varied for the subwatersheds upstream of the streamflow gage in an attempt to minimize the error between the observed and modeled results. However, while certain parameter combinations resulted in improved total discharge comparisons, each of these combinations adversely affected the peak outflow to such an extent that the original modeled hydrograph was preferable. Therefore, the final hydrologic basin parameters were unchanged from those imported from the original HEC-1 model. All design flows (stream flows corresponding to design storm rainfall inputs) were calculated using the basin-wide HMS schematic.

5.2.3. Improving the Hydrologic Model Input to the Hydraulic Model

Since the October 1994 storm event would also be used to calibrate the hydraulic model, it was desirable to improve the accuracy of the input to the hydraulic model by utilizing the observed streamflow data as a source input. Figure 5.10 shows the basin schematic used for generating streamflows used in the calibration runs of the hydraulic model. Utilizing the actual observed data removed any inaccuracies in the hydrologic portion of the study upstream of the gage. It is important to note however, that both the Marys Creek and Cowarts Creek subwatersheds drain to the hydraulic study region and
Figure 5.10: HEC-HMS Basin Schematic Representing the Hydrologic Model Using Stream Gage Input for the Upstream Subwatersheds

remain ungaged. Therefore, the HEC-HMS program must still model their corresponding flow contributions.

5.2.4. Calculated Stream Flows

Table 5-4 shows the resulting stream flows generated by the HMS model at four specific points throughout the hydraulic study region. These locations are presented as river stations in meters because available GIS data used in the hydraulic analysis of this report were only provided in meters. For clarification, a river station is the distance following the centerline of the natural, existing channel from a specified point downstream. In this case, the downstream point was defined as the downstream end of the hydraulic study region. A geographic representation of these locations is presented in later sections of this report.
Table 5-4: Resulting Stream Flow Rates (cms) for both the Design and Historical Storms

<table>
<thead>
<tr>
<th>River Station (meters)</th>
<th>100-Year Flow</th>
<th>25-Year Flow</th>
<th>October 1994 Flow</th>
<th>10-Year Flow</th>
<th>5-Year Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>10346</td>
<td>311</td>
<td>232</td>
<td>213</td>
<td>182</td>
<td>129</td>
</tr>
<tr>
<td>9141</td>
<td>395</td>
<td>298</td>
<td>275</td>
<td>239</td>
<td>171</td>
</tr>
<tr>
<td>8000</td>
<td>407</td>
<td>309</td>
<td>281</td>
<td>249</td>
<td>176</td>
</tr>
<tr>
<td>4778</td>
<td>543</td>
<td>413</td>
<td>362</td>
<td>343</td>
<td>241</td>
</tr>
</tbody>
</table>

It can be seen in Table 5-4 that the October 1994 flows fall between the 25-Year and 10-Year design flows as expected due to the rainfall intensities discussed previously. These flows were entered into the hydraulic model as flow data files used in the computation of the resulting water surface elevations.

5.3. Hydraulic Analysis and Results of Channel Modifications

A new HEC-RAS hydraulic model of a 6.5-mile reach of Clear Creek was successfully created using entirely digital topographic data in a GIS format and the HEC-GeoRas program. Chapter 4 addressed the specifics of the HEC-RAS program, its interaction with the HEC-GeoRas program, and the methodology for developing the required data sets for use in this model in significant detail. This section will focus on the results of the six geometric scenarios used to model proposed channel modifications to the hydraulic study reach passing through Friendswood, Texas. Some additional justification for the chosen channel modifications scenarios is provided. This is followed by a presentation of the resulting geometric data schematics representing the river reach in its pre and post-channelization state. Select cross-sectional elevation profiles derived
from a Digital Terrain Model (DTM) are compared to surveyed data supplied by Dannenbaum Engineering. A brief discussion of the modeling of the two bridges located in the study reach follows. The section concludes with a graphical representation of the resulting water surface elevations for each scenario.

5.3.1. Justification for Selected Channel Modification Scenarios

The six different channel modification scenarios discussed in Chapter 4 were chosen after discussions with hydraulic engineers in the Houston area familiar with Clear Creek and the unique flooding problems it presents. As discussed in Chapter 3, minimizing the amount of channelization throughout the watershed has long been identified as being of paramount importance for environmental as well as various hydrologic and hydraulic concerns. However, it was generally agreed in discussions prior to the commencement of this study that limited channelization schemes through already developed portions of the Clear Creek watershed should be investigated as part of any flood control solution (Dunbar, 2000; Flores, 2000).

The scenarios represent various channel widths and lengths that were preliminarily believed to be capable of accommodating design flows anywhere between the 5-year and 25-year flows. A 100-year design flow channel was considered too large and disruptive even when implemented on a limited basis throughout river reaches in the Clear Creek watershed (USACE, 1983). Varying the length of the channel modifications was performed to investigate if a short channel, channelizing only the most heavily developed reach of Clear Creek between FM 2351 and FM 528, would be able to accommodate at least a 10-year storm. One winding channel was to be investigated to
determine the effects of this non-straightened channel on water surface elevations. A winding channel would be beneficial compared to straighter channels due to reduced excavation costs and reduced environmental impacts on the riparian forest still present in the study reach. The additional base width of the winding channel was selected to compensate for the reduced hydraulic efficiency as a result of keeping the natural channel meandering. The various scenarios are again listed in Table 5-5 for convenience.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Condition</th>
<th>Resulting Reach Length / Percentage of Reach Modified</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Baseline / Existing (NC)</td>
<td>6.5 miles / 0.0%</td>
</tr>
<tr>
<td>2</td>
<td>Long, Widened Channel with 50-meter base width. No straightening. (LW50)</td>
<td>6.5 miles / 0.0%</td>
</tr>
<tr>
<td>3</td>
<td>Short, Widened Channel with 30-meter base width. Channel straightened. (SC30)</td>
<td>4.9 miles / 64%</td>
</tr>
<tr>
<td>4</td>
<td>Short, Widened Channel with 10-meter base width. Channel straightened. (SC10)</td>
<td>4.9 miles / 64%</td>
</tr>
<tr>
<td>5</td>
<td>Long, Widened Channel with 30-meter base width. Channel straightened. (LC30)</td>
<td>4.4 miles / 84%</td>
</tr>
<tr>
<td>6</td>
<td>Long, Widened Channel with 10-meter base width. Channel straightened. (LC10)</td>
<td>4.4 miles / 84%</td>
</tr>
</tbody>
</table>

5.3.2. Discussion of the Geometric Data Files Representing Channel Modifications

Figures 5.11 through 5.13 display the effects of the different scenarios on the hydraulic study region. Figure 5.11 shows a geospatially correct representation of the currently existing channel and the location and river stations of the cross-sectional elevation transects. This plan view of the geometric data file used to model Scenarios 1
Figure 5.11: Plan View of Existing Channel with No Modifications

Figure 5.12: Plan View of Long Channel Scenarios
and 2 clearly illustrates the additional reach lengths attributable to the natural meandering of the stream. Figure 5.12 shows a plan view of the geometric data file used to model the long channel Scenarios 5 and 6. Channel improvements (including straightening) have been implemented from the upper cross-section at river station 6873 until again joining the natural river meandering at river station 1585. It should be emphasized that cross-section stations and flow change stations change as a result of the straightening of the riverbed. Figure 5.13 shows the same information displayed in the previous figures except that channel improvements stop at river station 3704.

5.3.3. Elevation Cross-Section Comparison Between DTM and Surveyed Data

The HEC-RAS hydraulic model developed for this research was created completely from digital GIS information. As discussed in Chapter 4, all cross-sectional
elevation data for profile generation in HEC-RAS were extracted from a Digital Terrain Model created from a 10-meter resolution Digital Elevation Model (DEM). Preliminary research revealed that 30-meter resolution DEMs used in other hydraulic analyses provided insufficient topographic detail for accurate hydraulic results both inside and outside the channel banks.

Figures 5.14 and 5.15 show the comparison of DEM data versus surveyed data for two selected cross-section elevation profiles used by HEC-RAS in its hydraulic computations. The figures show an excellent match between the DEM data and the surveyed data outside the main channel banks. It is also clear from these figures the existing topography within the channel banks is poorly represented by the DEM. The cross-sectional elevation data could have easily been edited to match the surveyed data provided by Dannenbaum Engineering; however, it was decided not to perform these corrections to determine if accurate hydraulic results could still be achieved despite this discrepancy. Another reason for not correcting the DEM data was to determine the feasibility of applying this approach toward hydraulic modeling in areas where no survey information was available.

5.3.4. Water Surface Elevation Comparisons

The HEC-RAS program offers a variety of graphical methods to represent water surface elevation profiles along with a wide variety of other important hydraulic data such as bank elevations, energy gradient, wetted perimeter and stream velocity. This study focuses on comparing water surface elevations, but it should be noted that vast amounts of additional data results are created with theses particular hydraulic model runs.
Figure 5.14: Comparison between DEM and Surveyed Elevation Data at Station 8283 (No Channel Modifications)

Figure 5.15: Comparison between DEM and Surveyed Elevation Data at Station 7214 (No Channel Modifications)
Figures 5.16, 5.17 and 5.18 show the resulting water surface elevations for a variety of design storm flows and scenarios. Figure 5.16 shows a sample channel length profile of the 100-Year and 10-Year design storm elevations for the existing channel and the LW50 scenario. This method of data display is important for observing head losses through bridges and other hydraulic structures. The bridges are located at cross-sections 3600 and 5340 (existing condition river stations). The additional backwater elevation caused by friction losses through the bridges result in water surface elevation (WSE) increases from 0.1 to 0.2 meters. Again, it is important to note that the specific bridge data used for modeling their hydraulic effects was obtained from a previously existing HEC-2 file.

![Sample Channel Length Profile Displaying Water Surface Elevation Changes Resulting from the LW50 Scenario](image)

**Figure 5.16:** Sample Channel Length Profile Displaying Water Surface Elevation Changes Resulting from the LW50 Scenario
The downstream WSE boundary conditions are essential to the entire hydraulic modeling effort because they indicate the starting elevation and stream location where the hydraulic backwater calculations conducted in HEC-RAS commence. For consistency and accuracy, the boundary conditions were also imported from an existing HEC-2 hydraulic model. The boundary condition elevations for the 5-yr, 10-yr, 25-yr and 100-yr design storms were 3.46 m, 4.16 m, 4.54 m, and 5.15 m respectively. The boundary condition for the historical October 1994 event was estimated at 4.54 meters, equal to the 25-year design storm elevation. This approximation was necessary due to the lack of an actual known elevation point for the historical flood. Ideally, the hydraulic study would be conducted to the mouth of the stream, where water surface elevation is available in the form of Clear Lake elevation observations.

Figure 5.17 shows a cross-section profile from the LC30 scenario. This type of profile information can be used to determine the floodplain width at any specific WSE. The cross-sectional view also shows the 30-meter base width channel modification. Figure 5.18 shows a three-dimensional view of the water surface profiles and the two bridge crossings discussed earlier. This display also illustrates the effects of channel modifications to the previously existing channel morphology. Although this type of display is not directly useful from a quantitative standpoint, the three-dimensional representation gives an excellent qualitative feel for the extent of out-of-bank flooding. Additionally, this method of data presentation requires three-dimensional processing which was the precursor to many of the techniques used by HEC-GeoRas to develop digital floodplains.
Figure 5.17: Sample Cross-Section Profile Displaying Ground Surface and Water Surface Elevations

Table 5-6 shows the resulting water surface elevations at two locations along the hydraulic study reach for each proposed scenario.

Table 5-6: Comparison of Water Surface Elevations for Each Channel Modification Scenario

<table>
<thead>
<tr>
<th>Location</th>
<th>100-Year Design Storm Water Surface Elevations for Proposed Channel Modification Scenarios (meters, MSL)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Baseline</td>
</tr>
<tr>
<td>FM 2351</td>
<td>7.40</td>
</tr>
<tr>
<td>FM 528</td>
<td>6.29</td>
</tr>
</tbody>
</table>

The table confirms what was initially expected in that the long-channel scenarios more effectively reduce the WSEs at both locations along the stream. In contrast, while the
short-channel scenarios do have a noticeable impact on WSEs at FM 2351, their effects are drastically reduced at FM 528. Again, this is as expected do to the fact that channel modifications for the short channel scenarios end just downstream of FM 528 and the effects of this modification do not become substantial until further upstream.

Figure 5.18: Sample 3-D View of LC30 Channelization Scenario with Four Different Water Surface Elevations

The LW50 channel modification scenario had the least impact on WSEs at FM 2351. Based on the above, it was decided that only the long-channel scenarios had sufficiently reduced the WSEs at both locations. Therefore, only these two scenarios would be investigated further with the development of digital floodplains.

5.4. Calibration of the Hydraulic Model Using HEC-GeoRas
As is often the case in hydraulic analyses, the lack of an observed high-water mark during a storm event makes the calibration of any hydraulic model a difficult task. While an exact maximum WSE was not known for the October 1994 storm event, the significant number of Repetitive Loss Properties (discussed in Chapter 3) present in the Friendswood area allowed for their use as a general indication of the extent of flooding for any corresponding event. A repetitive loss property (RLP) database was obtained for the Friendswood area and the same procedure applied to the FEMA 100-year properties (discussed in Chapter 4) was applied to the RLPs.

As a result, a geospatially correct database was developed that showed the location of the FEMA designated 100-year floodplain properties as well as the location of the RLPs in the Friendswood area. This database provided two calibration opportunities for our hydraulic model. The FEMA properties provided a match goal for our 100-year design storm floodplain while the RLPs claiming flood damages in October of 1994 would serve as a match goal for the historical storm floodplain.

Calibration of the hydraulic model was attempted by adjusting the Manning's Roughness Coefficient along each of the cross-section elevation lines used to extract topographic data. HEC-GeoRas provides the ability to vary the coefficient along these transect lines. The coefficient is varied to reflect land use conditions in and around the channel of interest. This was performed for our hydraulic model. Figure 5.19 shows a land use map of the Friendswood area created by interpreting the 1-meter DOQQs discussed in Chapter 4. It is important to note that the DOQQs were created in 1996 and therefore, the land use file reflects 1996 conditions. A cursory site verification of the land use classifications was performed with satisfactory results. Categories used in the
land use map along with their corresponding Manning's Roughness Coefficient are shown in Table 5-7. The table also shows the coefficients used inside the channel banks. The coefficient varies with the state of the channel (modified, not modified, smooth and straight or rough and winding.) All Manning's values were obtained from Hoggan, 1997.

Figures 5.20 and 5.21 (included at the end of this chapter) show the October 1994 storm event and 100-Year design floodplains both prior to and after adjustment of the Manning's coefficient from a standard 0.045 (existing channel) and 0.035 (modified channel) to the coefficients shown in Table 5-7. A sensitivity analysis revealed that the WSEs and resulting floodplains were extremely sensitive to changes in the inner channel Manning’s coefficient.
Table 5-7: Manning’s Roughness Coefficients

<table>
<thead>
<tr>
<th>Land Use or Channel Condition</th>
<th>Manning’s Roughness Coefficient</th>
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</thead>
<tbody>
<tr>
<td>Forest / Woodlands</td>
<td>0.120</td>
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<tr>
<td>Agriculture / Pasture</td>
<td>0.060</td>
</tr>
<tr>
<td>Residential</td>
<td>0.050</td>
</tr>
<tr>
<td>Urban / Heavily Developed</td>
<td>0.030</td>
</tr>
<tr>
<td>Water</td>
<td>0.030</td>
</tr>
<tr>
<td>Existing Channel Upstream of River Station 6559</td>
<td>0.075</td>
</tr>
<tr>
<td>Existing Channel Downstream of River Station 6559</td>
<td>0.060</td>
</tr>
<tr>
<td>Modified Channel (LW50)</td>
<td>0.045</td>
</tr>
<tr>
<td>Modified Channel (All other scenarios)</td>
<td>0.035</td>
</tr>
</tbody>
</table>

However, as can be seen in Figure 5.20, adjustments of this parameter alone could not inundate all of the required properties for the historical October 1994 storm event. Prior to calibration attempts, 23 of the 60 October 1994 RLPs were in the modeled floodplain. After calibration, 39 properties were inundated. Of the remaining 21 properties not in the modeled floodplain, 15 are located in the upper portion of the hydraulic study region just upstream of the Clear Creek and Marys confluence, along Imperial Drive. It should be noted that flooding of these houses could be attributed to a variety of factors. Perhaps the most important of these is backwater effects along Marys Creek caused by its confluence with Clear Creek. Additional factors may include blockage of the primary and secondary drainage facilities.

The result of attempting to match the FEMA identified 100-Year floodplain properties is illustrated in Figure 5.21. Prior to calibration, the modeled 100-Year design
storm floodplain captured 150 of the 274 FEMA properties. After calibration, an additional 29 of the 274 properties were inundated resulting in 179 of the 274 properties in agreement. However, these modifications resulted in 83 additional properties being included in the modeled 100-year floodplain that were not considered inundated by FEMA's 100-year floodplain. These 83 additional properties brought the total to 262 properties within the modeled 100-year design storm floodplain.

5.5. Economic Comparison of Channel Modifications and Floodplain Property Buyouts

Chapter 3 presented a discussion of the various factors why non-structural flood control measures have historically not been implemented on a consistent basis. While a number of the possible explanations for this were discussed, one of the most important was the inability to accurately model these types of options and compare them with more readily modeled structural options (NWF, 1998). GIS databases containing property location and value information can be rapidly and easily combined with a HEC-GeoRas analysis to accurately determine the cost-effectiveness of the property buyout option.

Figure 5.22 (included at the end of this chapter) shows the simultaneous delineation of the four design floodplains analyzed in this report. These floodplains were combined with a property database that contained property and land value information to create dollar cost estimates for complete property buyouts of the properties within each respective floodplain. The land value database was generated from data available on-line from both the Harris County and Galveston County Appraisal Districts (HCAD, 2000; GCAD, 2000). Each of these organizations maintains updated plat maps that can be downloaded showing the location of all properties within a specified area. The property
value, improvement value, and land value for all structures within any specified area can be obtained by querying the on-line database with addresses obtained from the plat maps.

Table 5-8 shows the estimated cost for conducting a complete buyout of all properties within the listed floodplains. It should be stressed that this dollar value reflects the cost of purchasing all properties within the floodplains, irrespective of their flood damage history or slab elevation. As it is reasonable to assume that several of the properties in the modeled floodplains may have slab elevations higher than the design flood elevations, the dollar estimates should be regarded as high-end estimates. The dollar figures in Table 5-8 were obtained by conducting a GIS database query that requested identification of properties in the property database that were within the geographical limits of a particular floodplain.

<table>
<thead>
<tr>
<th>Floodplain</th>
<th>Number of Properties</th>
<th>Cost for Complete Buyout (Millions of Dollars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEMA 100-Year</td>
<td>274</td>
<td>35.2</td>
</tr>
<tr>
<td>Modeled 100-Year</td>
<td>262</td>
<td>35.0</td>
</tr>
<tr>
<td>Modeled 25-Year</td>
<td>138</td>
<td>19.0</td>
</tr>
<tr>
<td>Modeled 10-Year</td>
<td>87</td>
<td>12.2</td>
</tr>
<tr>
<td>Modeled 5-year</td>
<td>39</td>
<td>4.9</td>
</tr>
</tbody>
</table>

A complete listing of the properties identified within the FEMA delineated 100-year floodplain is included in Appendix D. This data set includes the address of the property, appraisal district lot number, its corresponding plat map number, the number of
National Flood Insurance Program claims made since 1977, the home and improved property value, the land value, and total property value for each property.

As discussed in Chapter 3, the combination of structural and non-structural flood control alternatives toward developing cost-effective, yet environmentally sound flood control plans is becoming increasingly more common throughout the United States. This combination approach was made possible in the Friendswood area by combining the previously discussed hydrologic and hydraulic models with accurate property cost databases.

Figure 5.23 (included at the end of this chapter) shows the effects of channel improvement scenario 5 (LC30) on the 100-year floodplain. Additionally, Figure 5.24 (included at the end of this chapter) shows the effect of this same channel on the 10-year floodplain. This scenario would create a long, straightened, 30-meter base width channel affecting approximately 84% of the original stream reach. The resulting, or residual 100-year floodplain after channelization, would still flood approximately 77 properties that would have to be bought out in order to provide complete protection against the 100-year design storm. Approximately 25 properties would still remain in the residual 10-year floodplain. Table 5-9 shows the total cost (in millions of dollars) for constructing an LC30 channel and vacating the remaining properties in the residual floodplain. The total expenditures shown for each design storm event would provide complete protection against these events. Approximately 679,000 cubic yards of material would have to be excavated to build an LC30 channel. Channel construction costs were estimated by using the excavation estimates provided by the CHIMP feature (discussed in Chapter 4).
available in the HEC-RAS program. The $2.4 million cost of excavation was arrived at by assuming a reasonable $3.50 per cubic yard excavation cost.

<table>
<thead>
<tr>
<th>Post LC-30 Floodplain</th>
<th>Number of Properties</th>
<th>Cost for Buyout (Millions of Dollars)</th>
<th>Cost of Excavation (Millions of Dollars)</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-Year</td>
<td>77</td>
<td>10.6</td>
<td>2.4</td>
<td>13.0</td>
</tr>
<tr>
<td>10-Year</td>
<td>25</td>
<td>2.9</td>
<td>2.4</td>
<td>5.3</td>
</tr>
</tbody>
</table>

Although figures for a Scenario 6 (LC10) channel modification are not included in this thesis for brevity, the research was completed with the results summarized in Table 5-10. It can be seen that although a greater number of houses remain in the residual 100 and 10-year floodplains when compared to the results of the LC30 channel, the significantly narrower channel would involve excavating less than half of the material excavated for the LC30 channel. The resulting reduction in excavation costs made the LC10 channel comparable to the LC30 channel with respect to overall cost if they were both combined with buyouts of the properties in the residual floodplains. Although not readily quantifiable, the narrower right-of-way associated with the LC10 channel may result in less of an impact to the surrounding riparian forest. Therefore, LC10 might be the most desirable channel modification. Further research would have to be conducted, however, before that claim could be fully substantiated.

Although the above economic comparisons are extremely rough, they are sufficient to show the effectiveness of combining both structural and non-structural
<table>
<thead>
<tr>
<th>Post LC-10 Floodplain</th>
<th>Number of Properties</th>
<th>Cost for Buyout (Millions of Dollars)</th>
<th>Cost of Excavation (Millions of Dollars)</th>
<th>Total Cost</th>
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</thead>
<tbody>
<tr>
<td>100-Year</td>
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<td>12.4</td>
<td>1.1</td>
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<tr>
<td>10-Year</td>
<td>35</td>
<td>4.5</td>
<td>1.1</td>
<td>5.6</td>
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alternatives in this situation and the effectiveness of digital floodplain databases. Perhaps one of the greatest advantages of these newer models is the relative ease with which they can be updated, modified and/or re-calibrated to reflect changing conditions or newly proposed flood control scenarios.

As an example, it is clear from the above results that implementing a channelization-only option would necessitate a much larger base width channel than 30 meters to completely contain the 100-year floodplain within the channel banks. The resulting required right-of-way would be much larger than the total 80-meter wide right-of-way strip associated with the LC30 channel displayed in Figure 5.23. A close look at the figure shows that even a slight increase in this easement would necessitate that several properties be relocated due solely to the size of the channel.

Lastly, although the most expensive of the presented options, the buyout-only option would provide several of the environmental and water quality benefits associated with the return of the natural floodplain. The reader is reminded that these benefits were discussed in Chapter 3. Quantifying these benefits is beyond the scope of this work, however, GIS technologies such as accurate land-use databases would likely assist a researcher in such an endeavor.
There exist an almost limitless combination of structural, non-structural and combination approaches to flood control. Only a few were closely investigated in this research. However, it should be clear to the reader that the digital databases and models developed herein would make the evaluation of these various alternatives a far more efficient and achievable task.
Figure 5.21

Modeled Floodplain Calibration Results for the 100-Year Design Storm

- Clear Creek
- Harris County Properties in FEMA 100-Yr. Floodplain
- Galveston County Properties in FEMA 100-Yr. Floodplain
- 100-Year Floodplain (Post-Calibration)
Effects of the LC30 Channel Improvement Scenario on the 100-Year Floodplain

Figure 5.23

- 77 Properties in the Post-LC30 100-Year Floodplain
- 262 Properties in the Existing 100-Year Floodplain *

Clear Creek
LC30 Channel Centerline
LC30 Channel and Right-of-Way
Post-Channelization (LC30) 100-Year Floodplain
Existing 100-Year Floodplain

* The 77 properties shown in red are a subset of these 262 properties.
Effects of the LC30 Channel Improvement Scenario on the 10-Year Floodplain

Figure 5.24

- 25 Properties in the Post-LC30 10-Year Floodplain
- 87 Properties in the Existing 10-Year Floodplain *

\( \text{Clear Creek} \)

\( \text{LC30 Channel Centerline} \)

\( \text{LC30 Channel and Right-of-Way} \)

\( \text{Post-Channelization (LC30) 10-Year Floodplain} \)

\( \text{10-Year Floodplain} \)

* The 25 properties shown in red are a subset of these 87 properties
Chapter 6. Conclusions and Recommendations

The latest HEC modeling programs, NEXRAD, and recently available GIS software such as HEC-GeoRAS have been effectively combined to begin addressing the flooding problems in the Clear Creek watershed using widely available digital data. Various channelization schemes and floodplain property buyout plans were modeled and analyzed. Preliminary results indicate that combining small-scale channelization with floodplain property buyouts serves as an effective flood control alternative while minimizing the drastic environmental impacts associated with large-scale channelization schemes. This novel, combined approach has permitted an accurate enough analysis to allow for the identification and quantification of individual properties in the floodplain. Although not currently a watershed-wide solution to flooding, the same approach can be implemented at other reaches along Clear Creek where flooding is extensive, given that the digital data used in this study is available for the entire watershed. As the digital data (DEM, DOQQs, etc.) used in this study become available at greater resolution and lower cost, this type of approach promises to become even more feasible. Further studies should include an analysis of error and its propagation through the various processes discussed in the approach.

Detailed objectives were listed at the beginning of this report. The conclusions of this research are listed below in conjunction with the objective with which they were associated. Recommendations for future studies and areas of improvement are also provided.
6.1. Conclusions for Objective 1

*Objective 1: Evaluate the overall flooding problem in the Clear Creek watershed and determine a suitable area of interest within the watershed for detailed study.*

The overall flooding problem in the Clear Creek watershed was investigated by reviewing several previous flood control plans developed both by the Corps and local government flood control districts. Flooding is a significant problem throughout the watershed and is attributable to a variety of factors discussed throughout the report. One of the most important at the watershed level is its geographical location and the resulting mild slopes, clay soils and periodically heavy rainfalls associated with the area. The inability to decide on any particular flood control option or combination of options to alleviate the watershed wide flooding problems has caused continuing flood losses through the 1990’s.

The stretch of Clear Creek through the city of Friendswood was chosen for a detailed hydraulic analysis for reasons discussed in the study. In summary, the area was chosen due to the rapid development in the area. The development has promoted floodplain encroachment and significant construction within the existing 25 and 100-year floodplains. It was determined that the local flooding problems in the Friendswood area were impacted by its proximity to two major confluences in the watershed.

6.2. Conclusions for Objective 2

*Objective 2: Utilize NEXRAD rainfall estimates for rainfall input into a hydrologic model of the Clear Creek watershed.*
NEXRAD radar was successfully used as rainfall input into a HEC-HMS hydrologic model by providing data for the October 1994 storm event in the area. The radar data compensated for the sparse location of rain gauges throughout the watershed. NEXRAD data was combined with GIS technology to develop individual subwatershed hyetographs that more accurately represented the storm event's rainfall intensity and distribution. Total storm rainfall amounts were also calculated for the watershed at the subwatershed level as well as the 1-kilometer square grid level.

6.3. Conclusions for Objective 3

Objective 3: Develop and illustrate the utility of a hydrologic model for the Clear Creek watershed in the improved Hydrologic Management System (HMS) format.

A HEC-HMS hydrologic model was developed by importing an existing HEC-1 model. Advantages of this updated model included ease of operation due to its Windows based format and the improved graphical user interface. The research showed that despite its recent introduction, the HMS program is ready for more widespread use. The map file feature of the program greatly facilitated the adjustment of individual subbasin hydrologic parameters. Perhaps the most important advantage that the HMS format exhibited over its predecessor was its ability to combine different basin and meteorological files in a rapid and efficient manner. The use of NEXRAD radar data as rainfall input to the model was validated by the modeled and observed streamflow hydrographs for the October 1994 event with acceptable results. The modeled peak flow for the calibration run was within 2.6% of the observed peak flow. The modeled time of peak was 2 hours prior to the observed for a hydrograph with a base time of over 9 days.
6.4. Conclusions for Objective 4

*Objective 4: Develop and illustrate the utility of a hydraulic model in the River Analysis System (RAS) format for determining water surface elevations in the study area.*

A new hydraulic model was developed for the reach of interest by utilizing GIS data and the HEC-GeoRas program. The model was created entirely from existing digital data with the exception of specific hydraulic structure data for two bridges in the reach. A 10-meter resolution DEM was converted to a TIN and used as the Digital Terrain Model for the hydraulic analysis. As was the case with HEC-HMS, the hydraulic analysis of the existing conditions and five different proposed channel modifications was greatly expedited by the Windows format and the graphical user interface. Two of the five proposed channel modifications were deemed to have sufficient impact on the water surface elevations to warrant further study utilizing a GIS-based alternatives analysis. The data display features of the program far exceed those of its predecessor, HEC-2. The program's ability to produce GIS export files was also successfully demonstrated.

6.5. Conclusions for Objective 5

*Objective 5: Utilize available GIS-data, ARCVIEW and the HEC-GeoRAS software to develop digital floodplains that can be used as both qualitative and quantitative measures of the effectiveness of various flood control options.*

Digital floodplain representations were developed using Hec-GeoRas at existing watershed conditions for the 5-year, 10-year, 25-year and 100-year design storm flows as
well as the October 1994 storm event flows. The resulting floodplain inundation areas were combined with a generated database of known flooded structures to assist in calibration of the hydraulic model. Floodplain mapping was accomplished by using ArcView. The process proved to be extremely cost-effective and efficient. The HEC-GeoRas / ArcView generated 100-year floodplain compared favorably with the latest FEMA 100-year floodplain in the area.

6.6. Conclusions for Objective 6

Objective 6: Combine the above models to investigate the effectiveness of two proposed flood control alternatives including limited channelization and floodplain property buyouts.

Four design storm floodplains were combined with a property database to approximate the dollar costs involved in performing complete floodplain property buyouts for each of the design floodplains. The estimated cost for buyout of all the properties in the 100, 25, 10, and 5-year floodplains was 35, 19, 12.2 and 4.9 million dollars respectively.

The effects of two proposed channel modification scenarios on the 100-year and 10-year design storms were modeled and graphically illustrated. Cost estimates were developed for constructing both a 30-meter and 10-meter basewidth long, straight channel. Neither channel was sufficient to accommodate all of the 10-year design storm flow, therefore, post-channelization residual floodplains were observed. Dollar cost estimates for constructing each of the channels and purchasing the houses in the residual floodplain showed that the both channel options were less expensive than the buyout only
alternatives. The reduced effectiveness of the smaller channel was compensated for by its smaller excavation costs. The developed hydrologic and hydraulic models, combined with generated GIS databases proved effective at evaluating these flood control alternatives.

6.7 Recommendations for Further Research

Based on the fact that a significant amount of this research involved developing hydrologic and hydraulic models in the most recently available HEC formats there exist a wide array of possibilities for further research. Additionally, the digital databases developed for economic comparisons in this report provide an excellent starting point for more in-depth hydrologic and hydraulic analyses. Some examples of further research are listed below:

- An updated digital land use database may be developed at the watershed wide level to attempt to develop a more accurate hydrologic model.
- The effects of the differences in the elevation cross-section profiles generated by the 10-meter DEM as compared to the surveyed data should be investigated more carefully.
- DEM data should be obtained for other Top 30 Repetitive Loss Property communities in the Clear Creek watershed so that similar research can be conducted in those areas. This would provide further validation to the results in this report.
- The next available significant rainfall in the area (over the 10-year design amount) should be used to validate the developed hydrologic model and provide additional needed calibration for the hydraulic model.

- A full and careful analysis of the propagation of error associated with specific portions of this study should be conducted when possible.
Bibliography


Appendix A: Galveston County Appraisal District
Property Information

<table>
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<tr>
<th>Address</th>
<th>Path</th>
<th>Room</th>
<th>Valuation</th>
<th>Market Value</th>
<th>Tax Value</th>
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### Appendix B: Harris County Appraisal District

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| **Total Harris County 2020 assessed valuation** | **$10,994,430** |