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Evaluation of Relative Hydrologic Effects of Land Use Change and Subsidence Using Distributed Modeling

by

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Abstract

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The present research employs remote sensing data and advanced hydrologic modeling to improve the spatial representation of an urban watershed, Whiteoak watershed, located northwest of downtown Houston, TX. During the last thirty years, upstream Whiteoak experienced rapid growth that locally altered its topographic and hydrologic parameters through land use change and subsidence. This study examines whether land use change and subsidence contributed to the increased flooding occurrences of the last decade in the area, amplifying the impact of moderate and extreme rainfall events. Detailed digital elevation data acquired from a 2001 Light Detection and Ranging (LIDAR) survey of Harris County, NEXRAD radar rainfall data, satellite land cover data, historical areal photos, and subsidence contours provide the main topographic and hydrologic parameters that support the analysis. All data sets are compiled in a GIS environment, processed, transformed into grid format, and imported into a physics-based, fully distributed hydrologic model, Vflo™. After setting up and calibrating the Whiteoak Vflo model, the analysis focuses on an upstream area located in the center of the subsidence bowl. Quantitative results indicate that continuing subsidence since 1978 did not significantly impact the local peak flows. On the other hand, the combined effect of upstream channel modifications and land use change over the same time period increased considerably the peak flows at the 100 year level.
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Chapter 1: Introduction

1.1 Urban Flooding

The present research applies distributed modeling techniques to identify and evaluate the causes of local flooding in an urban setting. In areas that receive annually significant amounts of rainfall, flooding occurrences are part of people’s lives. For the developing world, this is associated with the risk of several fatalities during major storm events. In developed countries, it usually bears the risk of significant financial losses. Among all natural disasters, flooding allows for some degree of prediction and preparation using present technologies and communication skills. Depending on the location, the direction, the intensity, and the duration of the storm events, the magnitude of flooding varies. But most importantly, vulnerability to flooding is strongly related to human action and behavior. This is the core idea behind the development of surface water management strategies and the underlying motive of the present study.

Water plays a vital role in life. During history, people chose to settle close to bodies of water, where land was fertile, transportation was easier, and most importantly, water was constantly available in desired quantities to support municipal, agricultural, and industrial needs. The present growth of urban centers experienced by most nations due to the movement of people from the rural areas to the cities, is mainly noticed in areas close to sea, rivers, or lakes. Inevitably, parts of the modern metropolitan areas are built in floodplains.

The 100-yr floodplain is a term commonly used in flood studies. A floodplain lies adjacent to a body of water and it can be occupied by water during a storm event, while under different meteorological conditions, it is part of dry land. A 100-yr floodplain is
associated with a 1% probability of occurrence storm event. TP-40, a nationwide technical report issued by the National Weather Service in 1961, provides maps that relate probabilities of occurrence with the rainfall volume and duration of an event (FEMA, 1995). Rainfall durations in these maps range from 30 minutes to 24 hours with return periods ranging from 1 to 100 years. The rainfall frequency maps are derived from the statistical analysis of point rainfall data from Weather Bureau gauges that possess 5 to 48 years of complete records (Hershfield, 1961). The TP-40 data are still used in studies that estimate the size of floodplains and decide on the appropriate structural and nonstructural measures that would reduce the flood hazards in a community. Some areas that continuously deal with flooding have conducted additional analyses that locally updated and adjusted the maps to include rainfall information from the last 50 years. The use of a probability value in the design of a surface water management plan carries with it the notion of an acceptable, affordable risk. From a government point of view, the Federal Emergency Management Agency (FEMA), founded in 1979, has the responsibility of evaluating risk and “developing an integrated emergency management system with an all hazards approach” that includes both natural hazard preparedness and civil defense activities (www.fema.gov , 2005).

Over time, the urban floodplain changes shape and size due to modifications of natural water pathways to accommodate development. Land cover roughness and imperviousness are affected by the new materials dominating the urban environment. Thus, a primary impact of urbanization is an increase in the runoff volume, due to limited infiltration, leading to an expansion of the floodplain limits. Other major impacts of urbanization are faster response time and increased peak flows due to a decrease of
surface roughness. In an urban setting, which involves concentrated human activities, the hydrologic effect of urbanization is significant and surface water management emerges as a requirement for human prosperity.

Five objectives of urban surface water management are suggested by Welsh (1989). First, and above all, the protection of human life and the reduction of risks related to public health and safety. Second, and very important, the decrease of financial risks related to private and public property. Third, the minimization of community affairs disruption, which affects people residing in and outside the flooded areas. Fourth, the protection of surface water and groundwater quality from potential pollutants carried by stormwater runoff. Finally, and most recently included among the objectives of urban surface water management, the enhancement of the quality of life by preserving, conserving, and restoring natural drainage features such as swales, channels, floodplains, ponds, and wetlands. The shift in the water control strategies over the years in the U.S to satisfy all the objectives listed above is shown in Figure 1.1 below:

Figure 1.1: Mission complexity versus changes in the water resources mission of the U.S. Army Corps of Engineers (National Resource Council, 1999)
What was not always understood in flood control planning, was the need for a greater master plan involving not only areas of immediate interest but also their upstream and downstream neighbors and taking as well into consideration the future development prospects of the community. The question of why there was a flood problem in the first place was not necessarily answered. The initial idea behind flood control measures was to route the water away from the affected area as fast as possible. Channel cross sections were locally enlarged while straight concrete channels replaced the naturally vegetated channel meanders. Detention basins were strategically built to hold excess water and reduce runoff volume during a rainfall event. In several cases, segmented planning of flood protection measures shifted flooding concerns from upstream to downstream areas, and resulted in channel erosion and sedimentation. Artificial protection from flooding led to a false feeling of security, more development in or close to flood prone areas, floodplain expansion, and more damage during later storm events. More damage called for more protection, which encouraged further development in flood prone areas (A.D. Little, 1973). A vicious circle was created.

Debo and Reese (2002) divided the reasons behind a flood occurrence into technical failures and deeper, institutional failures. Following that, they suggested the use of the “Five Whys” methodology, applied in total quality management, to reveal the connections between technical and institutional problems. Figure 1.2 on the next page illustrates the Five Whys approach with 6 blocks or “Levels”. The idea behind the Five Whys is that there are different levels of assessment of a complaint, so that each level can emerge from the previous one as an answer to the question “Why”. The physical and technical aspects of a flooding problem are included in the first three levels and can be
solved with structural, technical solutions. In many occasions, providing the technical solution leads temporally to complaint resolution and signifies the end of the analysis. However, there are often foundational issues beneath the obvious structural problems that allow for the existence of the hazardous situations. Debo and Reese recommend that possible root institutional problems be recognized and addressed in every case to avoid prolongation and repetition of similar flooding problems in a community.

**LEVEL 1**
Local residential flooding problems.

**LEVEL 2**
- Obstructed damaged structures
- High risk resident location
- Undersized structures
- Urban growth impacts
- A big flood

**LEVEL 3**
- Poor or no maintenance
- Ineffective development regulation
- Poor design criteria or data
- No urban growth assessment
- Little data collection

**LEVEL 4**
- Lack of technical knowledge and training
- Incomplete legal authority
- Insufficient funding

**LEVEL 5**
- Little public awareness
- Little political support

**LEVEL 6**
Lack of vision and direction

Figure 1.2 The Five Whys (Debo and Reese, 2002)
The consequences of lack of a comprehensive stormwater management program can be devastating in an urban community. On August 29, 2005, Hurricane Katrina made a landfall as a Category 4 hurricane in Louisiana and led to an unprecedented death toll and flood destruction in the country’s history. Among all the impacted areas, the city of New Orleans suffered the most, with an official death toll of more than 700 people, hundreds of thousand relocated, a few thousand people still missing, and 80% of the city under water during the peak flood. When the water eventually receded, significant contamination was left behind to be dealt with, rendering the city inoperable. The tragedy resulting from Katrina was the product of a fatal combination of technical and organizational inadequacies. As Waugh, Jr. (2005) points out, “Katrina was a man-made, nature-assisted disaster, resulting from a series of failures. Failure to preserve natural barriers to storm surge; failure to regulate development in the most hazardous areas; failure to adopt and enforce appropriate building standards and codes; failure to prepare residents for a catastrophic storm.” But the most significant failure was that no adequate action was taken to improve the community’s disaster resilience all these years, even though the region’s vulnerability had been clearly recognized in many significant studies conducted for Louisiana’s coastal areas.

Unique conditions supported the creation of a worse-case flood scenario in the city of New Orleans. This is rarely the situation in most of the cities lying in the flood prone areas of the U.S. However, the Katrina devastation proved once more how important is to identify and anticipate the hazards in an urban environment. The recent advances in computer science, in remote sensing technology, and in software allow for real time data collection and interpretation. As indicated in a report of the Subcommittee
of Disaster Reduction (2005) about the Grand Challenges of Disaster Reduction, it is important to invest in the creation of tools, advanced models capable of using the plethora of temporally and spatially varying data to improve natural process understanding and to provide reliable forecasting. Uncertainty abounds in all steps of natural process modeling. Many times, a disaster provokes a series of chain reactions, a domino-effect, triggering secondary events, and shifting risk from one area to another. It is important to develop and improve existing methodologies that analyze interdependencies among the different parameters of natural or man made systems and to identify the critical components among them. Distributed modeling has the potential to satisfy several of the previous requests and thus, holds the key to the future of hydrologic analysis and predictions. Based on this idea, the present study applies a distributed hydrologic model to isolate and weigh the relative contributions of different hydrologic parameters on the vulnerability of a fast developing urban community in Houston, TX.

1.2. Research Goal and Objectives

The current research uses advanced data and modeling techniques to address combined impacts of land use change and subsidence in a rapidly developing environment. With this thought in mind, the present study

1. Sets up a distributed hydrologic model for an urban watershed, Whiteoak watershed, located northwest of downtown Houston.

2. Calibrates and validates the model using significant historical storms of different intensities and durations.
3. Focuses on a smaller part of the watershed and quantify the hydrologic impact of spatially and temporally altered land use patterns.

4. Assesses the possible contribution of continuing subsidence to the increased flooding occurrences in the study area, and

5. Identifies the relative effect of land use change and subsidence on the floodplain of the study area.

The availability of remote sensing topographic and rainfall data supported the efforts of the study. On-site visits, discussions with competent authorities, and previous studies on the wider area facilitated the understanding of the past and present conditions and increased the motivation for a successful model application that enhances the understanding of underlying mechanics and provides input for the improvement of flood risk prediction and preparedness.
Chapter 2: Literature Review

2.1 Introduction

This chapter links the present study to developments related to urban hydrology, as documented in scientific literature. Emphasis is put on the latest technologic progress, which improved our ability to explore the spatially dependent physical processes governing hydrology. Water abundance and quality has always been closely related to the health and growth of communities. So, it seems natural that engineering hydrology practices exist in the findings of many ancient societies. From the evidence of hydraulic structures constructed by the Sumerians, the Egyptians, and the Chinese thousands years B.C., that served for irrigation and water management purposes, to the water systems of the Greek and Roman cities in the early A.C. years and the Incas hydraulic expertise at around 1000 A.C., the history of hydrology is an old one (Biswas, (1972), Crouch, (1993)).

Furthermore, the theory that supports the current hydrologic and hydraulic calculations of rainfall runoff has been in the literature for several decades. Bedient and Huber (2002) provide a summary of the early and the modern developments in the fields of hydrology and hydraulics. Rainfall and streamflow were first recorded during the 17th century. At the 18th century, Bernoulli’s theorem, the Pitot tube, and the Chezy formula set the basis for the development of modern hydraulics and fluid management. During the 19th century, Darcy’s Law, the Depuit-Thiem well formula, and the Hagen-Poiseuille capillary flow equation improved the understanding of groundwater flow. At the same time frame, the USGS began a systematic stream gaging that allowed for streamflow measurements on the Mississippi. The first half of the 20th century produced most of the empirical formulas and theories that support today’s hydrology. Sherman’s unit
hydrograph, Horton’s infiltration theory, Green and Ampt infiltration equations, Snyder’s Synthetic Unit Hydrograph were developed during that time. Shortly after the 1950’s, the Soil Conservation Service assumed relationships linking the direct runoff with the precipitation, and the infiltration plus the initial abstraction. Henderson and Wooding applied the concept of the kinematic wave to overland flow and streamflow calculations. Around the 1970’s, the first comprehensive hydrologic computer model, the Stanford model, was introduced. A few years later, the U.S ACOE Hydrologic Engineering Center developed HEC-1 and HEC-2 hydrologic and hydraulic computer models, which are the predecessors of the current lumped models HEC-HMS and HEC-RAS.

The advances in technology during the last ten years started a new chapter in the area of applied hydrology by expanding the capabilities of data acquisition and analysis. Remote sensing technologies provide detailed spatial and temporal data sets of precipitation, evaporation, infiltration, and topography. Powerful computers accept and handle the new data sets in a timely manner. New software is developed to link information with location. The modern tools support the development of distributed hydrologic models, improving in that manner the ability to assess the significance of spatially varying parameters that are interconnected in any hydrologic study.

Rainfall is the most important input, the necessary and sufficient condition for the existence of any flood related study. This thesis cannot but pay the proper respect to the significance of rainfall accuracy in hydrologic modeling by starting the literature review section with the advances in rainfall measurements and technology.
2.2 Rainfall data

The process of rainfall varies greatly with time and space resulting regionally to droughts and floods. From an engineering point of view, the accurate estimation of the rainfall distribution and volume is critical for evaluating, preventing, and mitigating the impact of different rainfall events. Traditionally, rainfall has been observed as point values provided by gauges. Nowadays, tipping bucket gauges are widely used in urban hydrology to produce rainfall depths and frequencies. Gauge rainfall data are interpolated to give estimates of mean areal precipitation of the entire basin or its subbasins.

However, several studies have demonstrated that the tipping bucket gauge suffers from systematic and random error. Groisman and Legates (1994) showed that point rainfall estimation errors range from 5% to 40% depending on the location and the time of year. One of the most important causes of gauge systematic error is the wind. In order to reduce the eddy currents from the wind, the U.S. Army Corps of Engineers (1996) suggested the placing of the gauges in protected sites with a wind shield and with consistent exposure away from shrubbery. Tipping bucket gauges could fail to tip during an event due to partial or complete clogging of the funnel that drains into the bucket, data transmission interruption, or temporary power failure. Guiliani et al. (1997) and Krajewski et al. (1998) discussed the techniques that can be used to detect and prevent the mechanical and electrical problems of the tipping bucket gauges.

Habib (2001) investigated sampling related errors of tipping-bucket rain gauge measurements, focusing on the gauge ability to represent the small-scale rainfall temporal variability. The results of this study showed that the gauge performance and its associated errors were sensitive to the applying sampling interval and the bucket volume.
In the beginning of 1990, the use of Doppler radar technology was introduced in hydrology. According to Doviak and Zrnic (1993), a sufficiently dense gauge network can measure rainfall better than a radar. However, such a network does not always exist and the outcome depends a lot on the patterns of rainfall. In addition, there is a considerable cost in setting up and maintaining a large number of recording gauges adequate to describe the spatial resolution of a storm event. Faures et al. (1994) examined the impact of the imperfect knowledge of spatial rainfall variability on a 4.4ha semiarid catchment. He compared the model runs he obtained using the information from all 5 raingauges in the study area to the runs he got using one raingauge at a time. The results of his study showed that even at that scale, during a convective storm, the single raingauge with the standard uniform rainfall assumption can lead to large uncertainties in runoff estimations, especially in smaller events. Brown and Bardsley (1998) investigated the use of a combination of recording and modified storage gauges to improve the description of individual rainfall events.

The development of powerful computers during the last decade, capable of handling huge amount of data in a timely manner, and the latest advances in radar technology resolved the issue of inadequate gauge networks. Radar data exceed the spatial densities of most rain gauge networks. Additionally, radar data provide temporal updates at intervals as short as five minutes. For both reasons, radar rainfall estimations are widely applied in water resources management, hydrologic design, and flood prediction and warning systems. Atmospheric conditions between the radar and the target affect the quality of the radar measurements. Variations in the rainfall speed and in the assumed rainfall drop-size-distribution add systematic and random errors to the
rainfall data. However, if calibrated by a gauge network, radar is considered to produce reliable rainfall estimates.

James et al. (1993) modeled the hydrologic response from the November 1986 storm on a 300 square mile watershed in Mississippi using as input calibrated weather radar data and precipitation gauge data. He concluded that in all streamflow hydrographs, the rising limb and peak discharge computed from the calibrated radar data are more accurate than those computed from the rain gauge data. Morin et al (1995) compared daily raingauge measurements with radar rainfall estimates using the window of probability matching method. The results of the study demonstrated good agreement between the radar rainfall and the raingauge rainfall. Mimikou and Baltas (1996) arrived to the same conclusion with James et al. after computing the hydrographs from six different storms, using as input in their hydrologic model calibrated radar data and rain gauge data.

Peters and Easton (1996) used NEXRAD rainfall data as an input to their HEC-HMS model and successfully simulated three storm events over a 4,163 km² watershed in Northwestern Oklahoma and Northwestern Arkansas. Seo et al. (1998) used procedures based on operational experience to estimate the mean field bias between the rain gauges and the radar data in real time in order to reduce systematic errors in rainfall data applied to hydrologic forecasting. Anagnostou et al. (1998) statistically studied the mean field bias of two-year Doppler radar observations using the associated raingauge measurements from a dense gauge network under the radar umbrella. The impact of varying gauge network densities was evaluated. The results of that study showed that the sampling error decreases proportionally to the square of rain gauge network density and
exhibits significant seasonal and time-scale effects. Johnson et al. (1999) compared 4000 pairs of mean areal precipitation values derived from NEXRAD stage III data and raingauge networks using Thiessen polygon weighting. They argued that NEXRAD generally tended to report 5% to 10% below gauged derived estimates with the exception of the smallest basin of their studies where the radar mean was higher than the raingauge mean.

Sun et al. (2000) calculated flood hydrographs for the Finniss river catchment in Darwin, Australia using different approaches to estimate the input rainfall from the available radar and rain gauge data. Their results showed that the cokringing of radar and rain gauge data improved flood estimates. Serafin (2000) argued about the present and future opportunities of WSR-88D. In his study, he discussed the advantages of the NEXRAD operational system associated with its quantitative and precise digital data. However, he recognized the existing problems related to accuracy of precipitation estimation, contamination of Doppler radar products by ground clutter, and the range of folding velocity data in regions of very strong shears. Droegemeier et al. (2000) focused on the significant flooding that can occur by tropical and extra-tropical cyclones pointing out that WSR-88D-based rainfall rate fields for tropical events are reasonably accurate.

Bedient et al. (2000) investigated whether a real time flood prediction system based on WSR-88D precipitation products could yield accurate and timely flood forecasts for an urban watershed. In their study, it was shown that NEXRAD-estimated rainfall performed as well as or better than the gauge estimated rainfall, despite large gauge radar biases. Considering that the radar could reach out to hundreds of kilometers beyond the watershed boundary as storms approached the area, the study concluded that NEXRAD
would become a valuable link in the creation of a flood warning system. Durans et al. (2002) evaluated whether the radar rainfall data can be used instead of raingauge data for the development of depth-area relationships, such as those published by the National Weather Service in TP40 and the NOAA Atlas 2. The study concluded, that shortness of data records and data heterogeneities were major obstacles to the development of radar based depth-area relationships at that time. Furthermore, the study was concerned over the biases in radar estimates of extreme rainfall events. Baun et al. (2004) recognized the uncertainties included in radar estimates due to the complexity of the mechanical and electronic systems of detection or due to the complexity of the terrain For this reason, he recommended an approach of calibrating radar measurements by applying mathematical methods proposed for image processing.

Vieux and Bedient (2004) used gauge adjusted radar data to reconstruct storm events in order to examine the achievable prediction accuracies of real-time operations in urban environments. The study demonstrated the need for bias-adjustment of radar in real-time to improve model accuracies to a satisfactory level. After obtaining gauge calibrated radar rainfall, the remaining random error diminished by the rainfall runoff process. Einfalt et al. (2004) focused also in the area of urban hydrology and the contribution of radar rainfall to urban watershed modeling. Their study provided a roadmap that identified the rainfall data requirements of urban hydrology, the basic differences of available weather radar types, the comparative strengths and weaknesses between radar and rain gauge data, the forecast potential and limitations of radar applications in urban areas, and the technical and organizational requirements of weather radar use. Furthermore, their paper included examples of online and offline radar
applications and ended with recommendations for successful radar data use in urban studies.

2.3 Hydrologic Modeling

Hydrologic models are classified into different categories according to the way they express the hydrologic processes. The present study models an urban watershed with the use of a physics-based, fully-distributed hydrologic model, Vflo\textsuperscript{TM} developed by Vieux and Associates at Norman, OK (Vieux, 2001 and 2004). A physics-based model applies the laws of physics, such as conservation of mass, momentum, and energy, to simulate the natural processes, in contrast to an empirical model that acts as a black box and describes rainfall-runoff through mathematical relationships. A fully-distributed model preserves spatial variability by dividing the domain into small interconnected cells. Model parameters are assumed to be homogeneous in each cell. On the other hand, lumped hydrologic models divide the watershed in larger parts, called subbasins, and most of the time, derive their parameters through optimization or empirical methods. Semi-distributed model cases exist, such as the TOPMODEL (Beven and Kirkby, 1979). TOPMODEL differs from either category by taking into account the statistics of its parameters along the basin while, at the same time, it is not interested on the spatial arrangement of the parameter’s values.

Major hydrologic models that vary in the way they handle space, time, processes, and solution techniques, are summarized and briefly reviewed in the hydrologic literature (Singh (1989), DeVries and Hromadka (1992), Bedient and Huber (2003)). The most widely used hydrologic model in the U.S. until today that deals with runoff from single
rainfall events, is HEC-1, designed by the Hydrologic Engineering Center in 1967 (HEC, 1990). HEC-1 is a lumped model that combines empirical and conceptual principles in its mathematical formulas. A Windows-based version of HEC-1, the HEC-HMS computer software package, was released by the Hydrologic Engineering Center in 1998, aiming to eventually replace HEC-1 (HEC, 2001). Detailed information over the HEC models and application examples are included in their technical manuals (HEC, (1990), HEC (2001)).

Berndtsson and Niemczynowicz (1987) argued that there was a gap between researchers and engineers in the field of hydrology. They encouraged the use of models capable of incorporating spatial rainfall variability to overcome the errors and uncertainties in the various hydrologic applications. The simplicity of lumped modeling and its limited data requirements, rendered its application in practical situations to look quite advantageous. However, distributed models' structure allows them to make the most of spatially varying data sets and take advantage of the increasing computer power.

The popularity of distributed models rose significantly with the recent progress in computer technology and data access. A decisive factor adding to the successful application of distributed modeling was the continuous improvement of radar rainfall accuracy. Removing errors from radar rainfall data sets responded to the concerns of various studies (Kouwen and Garland (1989), Doviak and Zrnic (1993)) that did not consider finer rainfall resolution as an improvement of the spatial quality of the rainfall data sets, when it unavoidably included various smoothing inaccuracies. The availability of suitable hardware and trained personnel, the decision of long-term commitment to the model, in-house model expertise, acceptance and support of the model; and commitment to modeling as a tool, were some of the factors that influence the choice of a specific
model in different cases (Moffa et al., 1990). Even though distributed modeling technology was considered as the technology of the future, more research was still needed to clarify the comparative strengths and limitations of lumped and distributed models.

For that reason, the National Oceanic and Atmospheric Administration’s National Weather Service (NOAA/NWS) launched a project on the year 2000 with the purpose of comparing the performance of current distributed models among them and to a lumped model used for operational river forecasting in the US. The Distributed Model Intercomparison Projet (DMIP) was designed to shed light on the following questions (Smith et al. 2004). First, whether distributed models improved the simulation accuracy compared to the lumped models. Second, what was the necessary degree of complexity to improve the basin outlet simulations. Third, how much effort was needed for distributed model calibrations and what were the results of that effort compared to non calibrated and calibrated lumped models. Fourth, whether distributed models calibrated for basin outlet simulations were capable to produce meaningful hydrographs at interior locations for flash flood forecasting. Fifth, whether there were distinguishing features that would establish a basin as a better candidate for distributed models than for lumped models, at basin outlet simulations. Sixth, how did research models perform with forcing data used for operational forecasting. And as a final question, what was the nature and effect of rainfall spatial variability in the DMIP basins.

Eight basins, ranging from 65 km² to 2,484 km² were selected for the study. These basins, while different in soil types and slopes, shared some common characteristics that made them good candidates for the DMIP project (Smith et al.,
They had the longest and highest quality of hourly NEXRAD data with archives starting from 1993. The quality of those rainfall data sets was tested in several previous studies. For the same time period, hourly discharge data was recorded at the basin outlets and at some interior points. In addition to that, the selected basins did not possess complications, such as orographic effects, significant snow accumulation, and modification of flows due to reservoirs.

Twelve models participated in the study following exact instructions for data use, calibration, and model runs. All but one models ran in continuous simulation mode. The r.water.fea model tested by the University of Oklahoma is an event based model (Vieux, 2004). The distributed hydrologic model used in this study is a commercial version of the r.water.fea re-written in Java™. An objective comparison between event-based and continuous models is hard to be completed due to the different manner that event models deal with initial moisture. For that reason, the DMIP study reported the event statistics from the University of Oklahoma model, but not any overall statistics. Reed et al. (2004) summarized the DMIP project results. The lumped model used in the DMIP study was the Sacramento Soil Moisture Accounting Model (Burnash et al. (1973), Burnash (1995)). The statistical comparisons between the models derived from the model runs of various storms events. For some basins, twenty-four events supported the analysis. For other basins, fewer events (down to sixteen minimum) were available for analysis. This was because observed flow records at the specific basins started at a later time.

Among the conclusions of the DMIP study reported in Reed’s paper were the following:
1. Some calibrated distributed models could perform at a level comparable to or better than the current operational standard lumped model. However, there were overall more cases in which the standard lumped model outperformed the distributed models. Model formulation, parameterization, and the skill of the modeler were significant factors influencing the model performance.

2. In most cases, calibrated distributed models outperformed uncalibrated distributed models during both the calibration and validation period.

3. Basins were identified that benefited more from distributed calibrated modeling due to their distinguishable shape, orientation, and soil.

4. The size of the basin was important for the relative inter model performance comparison. The DMIP study suggested that more research was needed to explain the relationship between model parameters and basin scale.

5. Models that combined conceptual rainfall-runoff techniques and physics-based distributed routing outperformed the other models during the calibrated tests, in all but the smallest basin.

6. Distributed models can be successfully applied to operational forecasting. However, since forecast precipitation has lower resolution, some of the gain coming from using distributed modeling may decrease for longer lead times.

Even though several questions about the performance of distributed hydrologic models were answered in that study, many more questions emerged. So, a Phase II DMIP study was suggested during the August 2002 workshop to continue research and improve the understanding of distributed hydrologic model structure and applications. In addition to the conclusions drawn from the models comparisons in the DMIP study, each modeler
separately examined his results, identifying reasons behind his model's different performance levels on the various basins and under different conditions. In their paper, Vieux et al. (2004) commented on the stability of the r.water.fea calibrated parameters during the different events. They pointed out that the very small change of the parameter values at the different storm simulations, and furthermore, the cases of even improved model performance at the model verification runs compared to the model calibration runs, supported their confidence in the model predictions and methodology. Another important observation in the same paper was that, at the interior point analysis, r.water.fea matched better the volume than the peak flow or the time to peak. Vieux et al. related that behavior with insufficient or inaccurate information in channel characteristics and routing.

Questions such as what scale of detail is needed for the analysis or what level of performance is required to satisfy the study goals, often precede the selection of a model. It has been argued that a distributed model of any scale, carries inadvertently a degree of lumping. The idea behind the selection of scale is identifying a maximum resolution beyond which, little additional information can be derived from our datasets (Vieux, 2001). One approach is to go for the scale necessary to portray the most rapidly changing parameter of the hydrologic model. However, there is always a trade-off between the scale size and the computation time. Seyfried and Wilcox (1995) pointed out the importance of correctly identifying the nature and sources of variability for reducing data requirements by focusing on the important sources. Depending on the case, it is possible that the impact of the variability of the most rapidly changing parameter is minimum to the goals of study, while it raises drastically the required computational time, especially
in larger watersheds. In those scenarios, a selection of a larger scale might be the appropriate modeling choice.

A factor that affects directly the scale selection is the watershed morphology. Costa (1987) examined the effect that catchment morphology and hydraulics have on the peak discharge of the twelve largest measured flash floods in the U.S. The basins of the study were smaller than 370 km². For his analysis, he tabulated different hydraulic and topographic characteristics of the basins such as hydraulic depth, hydraulic radius, Manning’s coefficient, channel slope, shear stress, mean velocity, and others. Costa argued that the force of the flood was dominated by the depth-slope product and added that an optimal combination of basin morphology, physiography, and storm intensity and not the greatest intensity duration events, were the reasons behind the creation of maximum flood peaks. Beven et al. (1988) used a modified version of a semi-distributed model, TOPMODEL, to investigate the existence of a threshold scale where the average hydrologic response is invariant. They called this scale a Representative Elementary Area (REA) and found that the size of REA is affected primarily from the topography. They recognized that the evaluation of the impact of catchment morphology in hydrologic modeling would be facilitated in future studies with the application of geographic information systems and the availability of digital elevation data. Black (1996) elaborated in his book the effects of different aspects of watershed morphology on flow and identified the watershed functions that dominated watershed hydrology. He emphasized that a good understanding of the role of watershed characteristics on runoff behavior would be essential for the implementation of successful water management practices.
2.4. The Use of Geographic Information Systems in Hydrology

The advancements in computer technology made it possible to deal with large data sets in a timely manner. There was a clear need for the appropriate interface to store, analyze, and display the detailed topographic and rainfall data sets, and prepare them accordingly to serve as input to hydrologic models. Geographic Information System (GIS) software filled that gap. Even though GIS applications existed since the 1960s, it was not until the early 1990s that GIS were employed in hydrology (Maidment and Djokic, 2000). The most important characteristic of GIS is its ability to associate information to location. GIS can deal with vector data (such as points, lines, and polygons) and raster continuous data. For each spatial entity, GIS stores its topology and attributes. A unique identifying number links the spatial features in a relational database.

In a GIS environment, data sets can be analyzed in different ways to serve different goals. Dodson (1992) suggested some of the most popular manipulations in GIS: a. Map overlays, in order to identify areas that share certain characteristics; b. Buffer generation, with the purpose to facilitate the identification of all features within a buffer area and organize them into tables; c. Boundary dissolve so as to develop new entities and coverages by regrouping and reclassifying existing entities, d. Tabular data analysis, in order to creates reports of tabular data from a database; e. Network analysis, with the purpose to identify convenient routes through networks; f. Digital terrain modeling so as to provide additional coverage of a study area, and g. COGO (coordinate geometry) in order to locate features accurately on a chart.

A very useful application of GIS in hydrology deals with terrain preprocessing and watershed delineation. Those processes are performed in GIS with the use of an
extension, HEC-GeoHMS, in combination with the Spatial Analyst extension. HEC-GeoHMS was originally developed as CRWR-PrePro by the Center for Research of Water Resources of the University of Texas at Austin (Hellweger and Maidment, 1999). HEC-GeoHMS is automated within a GIS environment to derive an ASCII file from digital elevation maps describing the various elements of a hydrologic model and their connectivity. The file is readable by the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS). Furthermore, the HEC-GeoHMS analysis of a digital elevation model produces grid layers describing flow direction and stream direction that can be imported into distributed models. Hellweger and Maidment applied the GIS watershed delineation at an Oklahoma watershed. Also, they tested the methodology at the Upper Mississippi basin and part of Missouri basin creating a lumped model that included more than one thousand hydrologic elements.

Since the first application of GIS in hydrology, there were several concerns about the limitations of its use in actual systems. Coppock (1995) identified five major issues regarding the use of GIS technology for natural hazards mitigation in past. First, there was a lack of comprehensive, detailed data sets. Several of the available data were coarse, incomplete, uncertain in terms of repetition, and very costly. Second, there were few models dealing with land surface that were capable of interpreting the varying and complex circumstances within a study area. Third, GIS technology, particularly that available in commercial systems, could not deal with the time dimension which is inherent in natural disasters. Fourth, GIS applications often could not meet the needs and knowledge limitations of their end users, when they were not scientists but city planners, policy makers, or emergency personnel. And as a final point, there were few supporting
organizations and incomplete infrastructure capable of translating the information obtained from GIS analysis to action.

Thoén (1995) collected information about the Internet resources related to the GIS and computer mapping. He acknowledged the fast growth of the Internet and the difficulties faced by the several researchers to keep up with that growth. In his paper, he provided a list of internet addresses concerning GIS resource documents, frequently asked question files, U.S. government organization websites with online data sets open to the public, software and data centers, and GIS workgroup mailing lists. Wilson et al. (2000) examined the continuous increase of number and variety of functions incorporated in GIS that were suitable for water applications. They emphasized the contribution of GIS technology to the accessibility and understanding of various data sets, the evaluation of promising solutions to water planning studies, and the development and implementation of more efficient hydrologic models. Garbrecht et al. (2001) summarized spatial data challenges common to many GIS-modeling applications. In their study, they presented sources, qualities, and limitations of data structures, projections, DEMs, digitized stream and drainage data, soil data, digital orthophoto data, and remote sensing data. Ogden et al. (2001) discussed models and hydrologic modeling applications that take advantage of GIS technology. Their paper pointed out critical issues arising from the transition of individuals and companies from homogenous hydrologic models to GIS based distributed hydrologic modeling.

Several hydrologic studies have been conducted using GIS either at the beginning of the analysis, in order to delineate the watershed and evaluate its parameters, or at the end of the analysis, with the purpose of displaying the computed hydrologic data on a

GIS data bases have been created for most big cities. The evolution of the cities in the boundaries of floodplains to facilitate transport, trade, and power generation, and to guarantee for water availability, has increased significantly the vulnerable to flooding population around the world. Urban flooding has become one of the most studied fields of hydrology.

2.5. Hydrologic Impact of Land Use Change in Urban Areas

Different cities have been studied to evaluate the hydrologic impact of development and to establish appropriate watershed management techniques that comply also with the existing legislation. The influence of land use change to the floodplain characteristics was always of great concern and several studies addressed it from financial, structural, managerial, or social angles. In 1986, Montz and Gruntfest updated a 1958 research from the University of Chicago related to the changes in American urban floodplain occupancy. In their paper, they examined 9 from the original 17 cities and evaluated the impact of policies and regulations to floodplain development. They noticed that annual flood damage figures rose despite the large investments in flood control works. Also, they observed that the reasons for increased floodplain occupancy leading to increased flood risk varied from community to community and suggested that more often
the National Flood Insurance Program succeeded where success was easier, while it was less effective in communities that faced development pressures.

New models, better data, and faster computers allowed for more precise analyses and quantification of the development effect on urban catchments. In studies such as Kim et al. (2002) in Indian River Lagoon watershed, urbanization was proven to increase the runoff volume up to 49% at some basins. In other studies, such as Ad De Roo et al. (2002) the Oder catchment, a balanced increase between forested and urban areas was considered the main reason that provided adequate flood risk protection in downstream reaches. Even rural watersheds that included man-made networks, such as ditches, roads, hedges, or underground drainage, showed a significant change on the hydrologic response of their basins. Carluer and De Marsily (2003) modified a distributed watershed model to evaluate the impact of human interference in a small watershed in Brittany, France, that contained grassland, trees, and simple man-made networks. Their work eventually aimed to assess the influence of development on the contaminant pathways. However, in their simulations, they showed at a small catchment scale, that by removing the man-made network, the peak discharges decreased, the recession became longer, and nearly all runoff would be surface runoff (after removing underground pipes).

The mild slopes and the proximity to the Gulf of Mexico, render Houston TX, susceptible to flooding. In addition to its topography and location, Houston experienced rapid growth during the last thirty years, that affected its natural drainage pathways and increased the inherent risk of flooding. Van Sickle (1962) derived unit hydrographs reflecting the effect of Houston’s initial transformation from undeveloped farmland to cultivated agriculture to urban setting on the direct runoff characteristics during the
period 1940-1960. Liscum and Massey (1980) and Liscum (2001) quantified statistically the hydrologic impact of Houston’s urban development on stormwater runoff characteristics. All studies agreed that urbanization significantly affected the shape of the rainfall hydrograph, increasing peak flow and reducing time to peak. However, there is always a concern that other factors related to urbanization, in addition to the changes in roughness, drainage network density, and geometry, enhance the severity of local urban flooding. In the Houston area more specifically, groundwater overpumping, has induced continuing subsidence rates that slowly change the local slope gradients over time. Subsidence could contribute to the increased flooding occurrences of the last few decades in Houston, locally amplifying the impact of moderate and extreme rainfall events.

2.6 The Effect of Excessive Groundwater Pumping

Among the issues attributed to urbanization is groundwater overpumping. The consequences of aquifer overexploitation are significant. Custodio (2002) mentions four points of great concern on having average aquifer abstraction rates greater than or close to the average recharge rates during an extended period: First, as the groundwater head drops, increased cost of development results from more energy consumption, early replacement and deepening of wells and pumps, and the need to enlarge the energy facilities. Second, progressive decrease in spring discharge, river baseflow, and surface area of wetlands would possibly compensate the difference between actual recharge and abstraction, leading to progressive depletion. Third, as the groundwater head potential changes along the wells or boreholes, or around a spring area, the abstracted water quality could deteriorate progressively due to different water mix. And finally, as the pore
pressure decreases, land subsidence will appear where sediments are unconsolidated. The size of subsidence can vary from few centimeters to many meters. As a consequence, the drainage pattern maybe modified and the afflicted area may become more flood prone and present signs of down-cutting erosion and coastline regression.

However, it has been argued by some (Lerner (2002), Kendy (2003)) that even though developing urban areas pump more groundwater per acre than irrigated rural areas, they actually increase the recharge in the aquifers. That is because most municipal water eventually returns in the hydrologic system, rather than evapotranspirating or been incorporated into products for export. Thus, it is expected that efficient domestic and industrial land use consume less water than irrigated cropland, mainly in unconfined shallow aquifers.

Lerner (1996) recognizes the unusual characteristics of an urban groundwater study. First of all, he admits that the recharge, which is affected by extensive sealing of surfaces, leaking water mains, sewers, and stormwater recharge, is often greater beneath cities than beneath equivalent rural areas. Then he puts emphasis on the large spatial variability of all processes involving the urban groundwater. Different recharge rates over short distances can be expected. Geotechnical interactions often occur, including aspects as interference with flow by extensive deep basements, tunnels, and piling. Local subsidence can result from extensive abstraction. The groundwater is affected by the point, multipoint, and linear input of chemicals from the multiplicity of urban features, such as industrial and residential sources, waste water systems, gardens, parks, and landfills. Because of the heterogeneity of land use and surface conditions in urban areas, chemical inputs are unlikely to occur that are uniformly distributed in space.
Management of groundwater protection is difficult and complex. Conflicts often exist between the economic need of different stakeholders of rapidly urbanized areas.

The issue of land subsidence due to fluid and gas overpumping has been recognized several years ago and evaluated in several studies (Poland and Davis (1969), Prokopovich (1972), Williams III et al (1987). According to Holzer (1999), more than 22,000 km² of land underlain by unconsolidated sediments in the U.S. has subsided over 30 cm owning to sediment compaction induced by groundwater withdrawal. A unique example of land subsidence related with urban development is the Houston-Galveston subsidence, which accompanied the vast exploitation of oil and groundwater in the area. Actually, Coplin and Galloway (1999) report that the subsidence of Goose Creek field between 1918 and 1926 at San Jacinto Bay was the first subsidence attributed to subsurface-fluid withdrawal to be described in the scientific literature. In that case, the low land became submerged, flooding and killing the vegetation, while eventually all peninsula disappeared beneath the water. Pratt and Johnson (1926) noted that the submerged volume amounted to the 20% of the produced volume of oil, gas, water, and sand. As much as 10 ft of subsidence have been recorded in Galveston Bay since 1906, resulting to extensive coastal flooding and inundation. Gabrysch (1984) studied extensively the spread of land subsidence in Harris County. He discussed the geology and hydrology of the region and explained the phenomenon of subsidence with several maps involving the principal areas of groundwater withdrawal, the pumping rates, and land subsidence. Especially close to the coastal areas, the negative effects of subsidence were recognized and groundwater pumping was minimized. However, subsidence shifted
inland following the development patterns of Houston as a consequence of extensive groundwater pumping for residential and industrial purposes.

Holdahl and Zilkoski (1991) modeled the regional subsidence patterns in the Houston area during the period 1973-1987. They also agreed that the regulated reduced pumping rates east of Houston have decreased dramatically the subsidence rates in that area. However, the rapidly growing West Houston area relies solely on groundwater pumped from the deep underlying aquifers and exhibits continued, increasing subsidence. Bravo et al (1991) analyzed also the subsidence in the Houston area using a 3-D finite difference model to simulate the hydrological conditions of the two underlying aquifers, Chicot and Evangeline. Their study aimed to couple the groundwater storage changes in the compressible beds with the aquifer system compaction. Potok (1991) was concerned about the impact of subsidence on the inland drainage system, in particular the riverine drainage system, the local drainage in small watersheds, and the Addicks and Barker Flood Control Reservoir, in Houston, Texas. His research included an evaluation of flooding as a result of subsidence along main drainage channels. His analysis concluded that there is a relationship between the depth of flooding and the change of channel gradient, with increases in gradient resulting to decreases in the flooding depth. On November 2001, the Fourth Conference of Aquifer Mechanics and Subsidence Interest Group took place in Galveston, Texas. The conference agenda included topics ranging from causes of subsidence and subsidence analysis to new technologies used for subsidence detection and monitoring and the development of regulatory plans (USGS, 2001). Carminati and Martinelli (2002) and Hu et al. (2004) investigated the appearance of land subsidence in Po Plain, Bologna, and Shanghai, China, respectively. In both
cases, subsidence was induced from water withdrawal. In the Po Plain study, a correlation was shown between subsidence and flood frequency by relating subsidence maps and flood maps.
Chapter 3: Research Methodology

3.1 Introduction

Chapter 3 provides information about the tools used for the collection and analysis of the data that support this study. The selection of the appropriate methodology to tackle the research problems is the most critical step taken after the main study goals have been identified. From the toolbox of theories and models, it is the researcher's responsibility to choose those that will allow him to shape general ideas into specific conclusions. Different criteria influence the choice of certain methods over others. The most important criterion is whether a specific methodology is sensitive (challenging) enough to reveal the contributions and correlations of the analysis variables to the questions posed. Some other factors are the availability of necessary software and hardware to support the methods, the availability of trained personnel to critically evaluate the output of the models, and the acceptance of the methods from the scientific community. Obviously, time and cost limitations are critical constraints in any project, even in a research study. Although more information and analysis are generally considered good, there is always a question whether the time and money spent result to proportional benefits from the moment that the initial research objectives are met.

Hydrologic studies need detailed spatial data to explain the phenomena modeled. Recent technological advances allow the access and handling of huge amount of digital data in an economically feasible and timely manner. Accordingly, the present research employs the latest technologies in the area of data mining, such as radar rainfall data, LIDAR digital elevation models, satellite land cover data, in order to describe as
accurately as possible the study domain. Subsequently, the data are assessed with the use of distributed modeling, capable of maintaining and analyzing large physical and topographic data sets for the elaboration of the various scenarios.

Figure 3.1: Flow chart of Research Methodology
As it is often stated, the results of a study can be at the most as accurate as the data set on which the analysis is conducted. For this reason, the role of appropriate data collection on the success of a research is enormous.

3.2 Data Collection

Rainfall data and elevation data are the two most significant driving forces of urban flood modeling. The intensity and duration of a storm event (and the resulting rainfall volume) are the criteria that define whether the event will be considered rare or not. In addition to that, the spatial variability of the storm over the watershed is closely related to the severity of flooding and the amount of damages on specific locations along the watershed, and increases the necessity of an operating flood alert system that would have provided lead time to establish adequate emergency response.

Besides rainfall structure, the volume of water, the peak flow, and the time to peak in a stream depend on the changes in slope, roughness and imperviousness (due to different land use practices and soil characteristics) along the watershed. Thus, accurate topographic data are essential, especially in watersheds with significant variations in slopes. Houston TX, is characterized by mild slopes that increase the travel time of storm water and reduce the peak discharge of storm events. However, Northwest Houston is experiencing over the last 30 years continuing, localized, and significant subsidence rates that alter the elevation differences between adjacent areas. In order to examine whether the local changes in slopes add locally to the increased flooding events of the area, detailed topographic data capable of depicting the changes of elevation due to subsidence are needed.
Because of the clay or sandy clay soil, there is little infiltration along Houston watersheds. Thus, the impact of the rapid urban development of Houston during the last few decades on infiltration, as a result of change in the imperviousness, is rather small. In addition to the imperviousness, land use change affects also the roughness of the watershed and subsequently the peak flow and the time to peak. Satellite land cover data offer essential, thorough information about the present land use patterns in Houston.

The following three sub-sections provide background theory about the collection of the three data sets, rainfall, elevation, and land cover, that are used in the study.

3.2.1 Radar Rainfall Data

The proximity of Houston to the Gulf of Mexico renders it susceptible to heavy rainfalls. For this reason, the rainfall monitoring gauge network of central Houston watersheds is considered dense compared to the gauge networks of other cities. Whiteoak watershed, that drains an area of approximately 110 mi², contains 9 rain gauges inside its basin and has 9 rain gauges close to its boundaries. Nevertheless, studies have shown, that convective storms (commonly affecting Houston) often include pockets of heavy intensity rainfall that are not captured from the gauges. In addition to that, during large storm events, there is a good possibility of some gauge malfunctioning. In that case, the ability of capturing the spatial variability of rainfall is hindered and could result to significant error in rainfall calculations.

Radar technology started to evolve a little before World War II. However, it was not until recently, that advances in computer technology allowed for timely handling of large data sets and supported the development of software to deal with the data plethora.
In parallel, continuing improvements of radar technology either in the area of rainfall detection or in the area of spatial and temporal data resolution and analysis, further encouraged the use of radar data in hydrologic simulations.

![National Doppler Radar Sites](image)

KHGX-Houston/Galveston, TX

Figure 3.2 (Left): The NEXRAD Network, obtained from the NOAA website (2005); (Right): Prototype Radar at the National Severe Storms Laboratory, Norman, OK

The rainfall data of this study are obtained from the KHGX-Houston/Galveston Doppler radar, located at Dickinson, approximately 40 km southeast of downtown Houston. With a range of 230 km, it provides a good coverage of the study area. A conventional radar looks like a parabolic dish and emits a pulsed beam which travels through the low atmosphere. When the wave strikes an object (raindrop, hail, insect, etc), it gets scattered in all directions. Some part of it is reflected back to the radar, which processes it and presents it with a color display. A Doppler radar can additionally detect the movement of the particles, taking advantage of the "Doppler Effect", the shift of frequency of the reflected signal. A brief explanation of the Doppler effect is given
below. A radar emits a radio wave at a specific frequency. If both radar and target are
still, the reflected signal will have the same wave frequency as the original signal.
However, if the target (a raindrop in our case) is moving, each part of the radio wave will
be reflected at a different point in space. If the target moves towards the radar, the
reflected wave will be compressed resulting to higher frequency, whereas, if the target
moves away from the radar, the wave stretches out and its frequency becomes lower.

The name NEXRAD (NEXt generation RADar) is commonly used to describe the
Doppler weather surveillance radar WSR-88D, prototyped by the National Severe Storms
Laboratory in Norman, Oklahoma (Figure 3.2). The NEXRAD network in the United
States, installed during the 1990’s, provides a full coverage of most areas, with the
exception of some Western parts, where either the terrain morphology, or the lack of
radars is the cause of some rainfall detection gaps.

The pulse repetition frequency of a radar is 325 pulses per second. The radar
antenna rotates starting from a 0.5 degrees tilt. After each revolution, it increases the tilt
by 1 degree in order to scan the entire surrounding atmosphere. Rainfall rate R, is
determined from the reflectivity (the amount of reflected power returned to the radar after
hitting the raindrops) values. Reflectivity Z, is measured in a logarithmic scale known as
the decibel scale. The scale ranges between 5 and 75 dBZ when the radar works in the
precipitation mode. Higher values represent more reflected power to the radar. Moderate
rain is between 30 and 45 dBZ. Both reflectivity and rainfall rate can be expressed related
to the median drop size. The reflectivity and rainfall rate equations can be combined to
form a reflectivity-rainfall rate relationship. Depending on the season and on the
particular storm type, there are two equations generally used to derive rainfall rates. The
standard Z-R equation, \( Z = 300R^{1.4} \), is the one installed in all the Doppler radars (Vieux and Bedient, 1998). However, for tropical storms, the use of the Z-R relationship \( Z = 250R^{1.2} \) is recommended (NOAA-NWS, 1995).

Before using the Z-R relationships, algorithms are applied to the reflected data to remove spurious effects caused by sidelobe contamination, ground clutter, and anomalous propagation of the beam. Different weather effects, such as downdrafts or updrafts during a thunderstorm, can locally affect rainfall rates, vary the assumed drop distribution, and add to the systematic and random errors of the radar rainfall estimation. However, the systematic errors can be removed by calibrating radar rainfall data with the rain gauge data. At each gauge vs. radar rainfall comparison, radar rainfall accumulations may underestimate or overestimate gauge rainfall accumulations. A Multiplicative Factor, \( F_{RM} \), is identified as the ratio of the mean gauge estimations divided by the mean radar (Vieux, 2002). Radar systematic error is corrected by multiplying the radar data with \( F_{RM} \) and thus, removes the mean field bias of radar rainfall.

\[
F_{RM} = \frac{\sum_{i=1}^{n} G_i}{\sum_{i=1}^{n} R_i},
\]  

(3.1)

where \( G_i \) and \( R_i \) are the gauge/radar pairs of rainfall accumulation and \( n \) is the number of pairs used for the multiplication factor calculation. After the bias correction, the calculation of the Relative Dispersion provides an estimate of the cluster of data around the mean while an estimation of the random error is succeeded by the calculation of the Average Difference statistic (Vieux, 2002). The mathematical definitions of Relative Dispersion is given by
\[ RD = \sigma\left(\frac{G_i}{R_i}\right) \]  \hspace{1cm} (3.2)

where \( \sigma \) is the standard deviation of the ratio. The Average Difference \( D \) after the bias correction is,

\[ \overline{D} = \frac{100\%}{n} \sum_{i=1}^{n} \left| \frac{G_i - F_{RM} \ast R_i}{G_i} \right| \]  \hspace{1cm} (3.3)

The precipitation products supporting this study have been processed and gauge adjusted by Vieux and Associates Inc. at Norman, OK. Three significant storm events for Whiteoak watershed have been selected for the model calibration and the hydrologic simulations: September '98 storm event (Tropical Storm Frances), October '02 storm event, and November '03 storm event.

3.2.2 LIDAR Digital Elevation Model Data

LIDAR stands for Light Detection And Ranging technology. Soon after Tropical Storm Allison severely hit Houston in June 2001, FEMA and the Harris County Flood Control District initiated a program called Tropical Storm Allison Recovery Project (TSARP) that aimed to improve Houston's resilience to violent weather events. Part of that commitment to the community was the application of LIDAR technology to enhance the quality of existing topographic data sets of the Houston wider area (TSARP, 2002).

LIDAR is a remote sensing technology that uses laser light to collect topographic data. The National Oceanic and Atmospheric Administration (NOAA) and NASA have been using LIDAR to evaluate the changes of shorelines along time. In airborne LIDAR, a laser sensor and a GPS device are mounted on an aircraft or a helicopter. During flight, the laser emits thousand pulses per second at a near infrared frequency, which is invisible
to the human eye. When a pulse meets the ground, it is reflected back or scattered by the
target. The flight elevation is approximately 3,000 ft. Thus, it takes only about 6.1x10^{-6}
seconds from pulse emission to detection. The size of the pulse at the ground level is
about one ft in diameter. Depending on the land use and slope of the area, up to 5 returns
can be recorded per pulse. The surface elevation is determined from the time to travel of
the pulse. The recorded time to travel is associated with the position and orientation of
aircraft through the GPS instrument. The collected data points are post-processed in
ASCII or text file formats, where each point is described by its x,y, and z coordinates.
These files are large and not easily manipulated. However, they can be interpreted in a
regular Digital Terrain Database, which includes the topographic surface with its
significant topographic features (such as breaklines and spot elevations). From a Digital
Terrain Model, a Digital Elevation Model can be derived by removing these topographic
features.

The first DEM product needs several hand-edits to represent realistically the
surface of an entire project. It is known that LIDAR technology performs less accurately
in areas with tall grass, with overhanging trees, in channel cross sections with water, and
in areas paved with asphalt. For the improvement of the Harris County LIDAR project,
five thousand cross sections were conventionally reviewed by 5 independent contractors.
In addition to that, the LIDAR data were checked for smoothness, while identified critical
points were evaluated with the help of aerial photography and were adjusted against
previous conventional field surveys. The data points initially collected for Harris County
were at 1.5-meter horizontal intervals with 0.15 cm surface accuracy. From those points,
a 5-meter DEM was derived on a State Plane Texas South Central projection and is
available from the TSARP web site. The elevation units are decimal feet. Figure 3.3.a. on the left illustrates the process of airborne LIDAR mapping. On the right, Figure 3.3.b. shows a detailed representation of a section of the Whiteoak watershed terrain using LIDAR technology.

![Figure 3.3: LIDAR technology application. a: Schematic of airborne LIDAR mapping retrieved from http://www.emporia.edu/earthsci/student/serr1/project.htm (2006) b: LIDAR representation of Whiteoak bayou near I-10 retrieved from the TSARP website (2005)](image)

3.2.3 Satellite Land Cover Data

The land cover information used in this study was obtained from the USGS web site and represents imagery and land cover data based on Landsat Mapper satellite data from the period 1992-1995. The land cover data set was a part of a greater project between USGS and the U.S. Environmental Protection Agency (USEPA), as mentioned at the National Land Cover Texas Southwest Report (NLCD Texas Southwest, 2000). The NLCD report states that the purpose of the project was to produce a consistent land cover data layer for the conterminous United States based on 30-meter Landsat thematic mapper (TM). The base data set was leaves-off Landsat TM data, acquired at 1992.
Additional data sets used in the project included leaves-on TM, USGS 3-arc second Digital Terrain Elevation Data (DTED) and derived slope, aspect, and shaded relief, Bureau of the Census population and housing density data, USGS land use and land cover (LUDA), and National Wetlands Inventory (NWI) data, if available. All data sets were projected in the Albers Conical Equal Area Projection. The 21 classes used to explain the spatial variability of land cover are summarized in table 3.1 below:

<table>
<thead>
<tr>
<th>Class</th>
<th>Number</th>
<th>Class</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td></td>
<td>Shrubland</td>
<td></td>
</tr>
<tr>
<td>Open water</td>
<td>11</td>
<td>Shrubland</td>
<td>51</td>
</tr>
<tr>
<td>Perennial Ice/Snow</td>
<td>12</td>
<td>Non-Natural Woody</td>
<td></td>
</tr>
<tr>
<td>Developed</td>
<td></td>
<td>Orchard/Vineyard/Other</td>
<td>61</td>
</tr>
<tr>
<td>Low Intensity/Residential</td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>High Intensity/Residential</td>
<td>22</td>
<td>Herbaceous Upland</td>
<td></td>
</tr>
<tr>
<td>Commercial/Industrial/Transportation</td>
<td>23</td>
<td>Grasslands/Herbaceous</td>
<td>71</td>
</tr>
<tr>
<td>Barren</td>
<td></td>
<td>Herbaceous/Planted/Cultivated</td>
<td></td>
</tr>
<tr>
<td>Bare Rock/Sand/Clay</td>
<td>31</td>
<td>Pasture/Hay</td>
<td>81</td>
</tr>
<tr>
<td>Quarries/Strip Mines/Gravel Pits</td>
<td>32</td>
<td>Row Crops</td>
<td>82</td>
</tr>
<tr>
<td>Transitional</td>
<td>33</td>
<td>Small Grains</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fallow</td>
<td>84</td>
</tr>
<tr>
<td>Forested Upland</td>
<td></td>
<td>Urban/Recreational Grasses</td>
<td>85</td>
</tr>
<tr>
<td>Deciduous Forest</td>
<td>41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Evergreen Forest</td>
<td>42</td>
<td>Wetlands</td>
<td></td>
</tr>
<tr>
<td>Mixed Forest</td>
<td>43</td>
<td>Woody Wetlands</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Emergent Herbaceous Wetlands</td>
<td>92</td>
</tr>
</tbody>
</table>

Table 3.1: NLCD land cover classification system key (NLCD, 2000)
3.2.4 Historical Aerial Photos

For the purposes of this study, historical aerial photos of the upper Whiteoak watershed were purchased from a local aerial surveys incorporation. These photos were from the years 1978 and 1987 in black and white format and had resolution 1 to 3000 ft. In addition to these photos, color aerial photos of Whiteoak watershed from the year 2000 were available at 0.5 meters resolution from the Fondren Library/GIS Center at Rice University.

3.3 Data Analysis

After the collection of data, the attention shifts on the selection of appropriate software tools to analyze and evaluate these data sets in a meaningful way. Two software packages are mainly used in the current study: 1) ArcView, a user friendly desktop Geographic Information System software developed by ESRI and 2) Vflo, a physics based, fully distributed hydrologic model. The two models can exchange information with each other. The gridded input data required from Vflo are prepared from the available data in ArcView, while different hydrologic parameters used in Vflo (e.g. storm) can be exported at any point of the hydrologic modeling back to ArcView for further spatial visualization and further analysis. The next section provides a brief description of ArcView abilities while more information about Vflo is given in part 3.3.2.
3.3.1 GIS – ArcView

As mentioned in Chapter 2, the Geographic Information Systems tools were applied in the area of hydrology in the early 1990s even though they had been used in other fields for over 40 years. GIS is a mapping system that assembles, stores, spatially interprets, and displays data. In its desktop application, ArcView, the spatial information is arranged in Themes. There are vector and raster Themes. A vector Theme describes various features such as gauges, rivers, buildings, with points, lines, or polygons. A raster Theme is based on individual cells (or pixels), where the feature attributes (such as color, elevation, id) are considered homogenous within a pixel. Each vector Theme is connected with a Table of Attributes that can include geographic or non geographic data about the various features. The workspace where Themes are analyzed is called a View. An ArcView Project can contain several Views that analyze various Themes and their Attributes in different ways. The spatially displayed information can be exported or printed from ArcView in the form of maps through the Project Layouts.

Specially designed extensions have been created to support different goals in the scientific community. This study widely uses the HecGeoHMS, the Spatial Analyst, and the Projection Analyst extensions to prepare the various data sets for the hydrologic modeling. Figure 3.4 at the next page shows a View of a Whiteoak GIS project that includes raster Themes (the digital elevation model of Whiteoak) and vector Themes (the streams, the roads, the gauges). For illustration purposes, Figure 3.4 also displays the table of Attributes of a specific road and the tags that identify highways and gauges.
Figure 3.4: A View from the Whiteoak watershed ArcView project

3.3.2. Vflo Distributed Hydrologic Model

The rainfall-runoff simulation of the current study is conducted with the use of a physics-based, fully distributed hydrologic software, Vflo™ (Vieux and Associates Inc., 2004). Vflo was developed by Vieux and Associates Inc. at Norman, OK and is based on finite element analysis. It is computationally efficient and capable of simulating historical rainfall-runoff events and operate in real time, at any location of the study domain. As an input, Vflo uses grid data layers developed in GIS, affording the desired spatial accuracy and versatility to model rainfall runoff in large and small basins (Vieux, 2004). The graphic interface of Vflo is a grid with a defined drainage direction for its various cell types, as shown in Figure 3.5 on the next page:
Figure 3.5: Vflo Graphic User Interface for Whiteoak basin: shapefiles of boundaries, cross sections, and highways facilitate the basin understanding; Zoom-In detail: blue arrows stand for channel flow direction and green arrows for overland flow direction.

All data layers imported in a single study must adhere to the same grid resolution and size with the decided dimensions of the study basin grid. Vflo calculates runoff using six different cell types: overland, channel, rated channel, cross-section, reservoir, and base cells. Thus, runoff at each cell depends both on cell type and cell properties. Overland flow is the result of rainfall rate exceeding soil infiltration rate and is conceptualized as thin sheet flow. The flow and depth of channel flow is directed from the hydraulic characteristics of the channels. Rated channels use rating curves (area-stage and stage-discharge) developed from field measurements and hydraulic modeling to calculate flow. Cross-section channels require ordered pairs of distance and elevation to describe the channel geometry and calculate the stage-discharge relationship. Reservoir
cells contain initial storage value and rating curves that describe how flow propagates through the cell. Base cells are the simplest types of cell that can be used in a Vflo model. They require only flow direction. Any flow that passes through those cells, will move to the next one based on kinematic wave celerity. It is recommended to use base cells only to describe shallow water areas in wetlands and marshes or lakes with long shallow water upper reaches that extend through several cells (Vieux, 2004). Figure 3.6 below describes schematically the runoff process in Vflo:

![Diagram](image)

Figure 3.6: The grid cell runoff process in Vflo

Vieux (2004) describes thoroughly the analysis and equations applied in Vflo and the reader is directed to that text for a clear explanation of the methodology. Some basic points included in that source are summarized below. Vflo uses 1-D linear elements to simulate overland and channel flow. Their connectivity depends on the drainage direction map derived from the analysis of the Lidar DEM in the GIS environment. In contrast to
conceptual-mathematics representation hydrologic models, Vflo applies the equations of conservation of mass, momentum, and energy to explain runoff generation and routing.

The 1-D momentum conservation equation is described as,

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \left( \frac{\partial y}{\partial x} - S_o + S_f \right) = 0 ,$$

where $S_f$ is the friction slope, $S_o$ is the bed slope, $V$ is velocity, $y$ is the hydraulic depth, $x$ is the distance along the path, and $t$ is the time. Rearranging the terms, $\frac{\partial y}{\partial x}$ represents the pressure gradient, $\frac{\partial V}{\partial x} \cdot \frac{V}{g}$ represents the convective acceleration, and $\frac{\partial V}{\partial t} \cdot \frac{1}{g}$, the local acceleration. If we assume uniform flow, local acceleration, convective acceleration, and pressure gradient can be considered insignificant. Then, the equation of conservation of momentum can be simplified to the kinematic wave analogy (KW),

$$S_o = S_f$$

Under the kinematic wave assumption, flow depends solely on depth. Also, the slope of the water surface and the friction slope are taken parallel to the land surface slope. KW does not allow for backwater calculations. Radar rainfall or gauge rainfall is sampled for each cell comprising the Vflo hydrologic model. The 1-D continuity equation (or the equation of conservation of mass) for overland flow resulting from rainfall excess is

$$\frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} = R - I ,$$

where $R$ is the rainfall rate, $I$ is the infiltration rate, $h$ is flow depth, and $u$ is the overland flow velocity.
For channel flow, 1-D continuity equation can be expressed in terms of the cross sectional area \( A \):

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q ,
\]

where \( Q \) is the flow rate in the channel and \( q \) is the rate of lateral inflow per unit channel length.

Under the uniform flow assumption of kinematic wave approximation, Mannings equation relates velocity and flow depth:

\[
u = \frac{S_0^{1/2}}{n} h^{2/3}
\]

When equation 3.8 is applied on 3.6, it results to a relationship between flow depth, landsurface slope, hydraulic roughness, rainfall, and infiltration:

\[
\frac{\partial h}{\partial t} + \frac{S_0^{1/2}}{n} \frac{\partial h^{5/3}}{\partial x} = R - I
\]

The partial differential equations (3.9) and (3.7) that depend on space and time are transformed to ordinary differential equations in time with the application of finite element analysis. Galerkin approximation is chosen to lead to the analytical solution of the differential equations. A finite-difference scheme is applied to solve the time-dependent system. Since 1-D elements are chosen, a \( w_{eq} \) elemental width is introduced to account for the correct representation of the drainage area. In the case where the watershed flow elements lead to one outlet, there is no bifurcation in the downstream direction at any cell, and the channel area is a large fraction of the total area, \( w_{eq} \) equals to:

\[
w_{eq} = \frac{Area_{total} - Area_{channel}}{Length_{total} - Length_{channel}}
\]
When the model is set up, Vflo uses scalar multipliers to adjust the maps representing critical parameters such as roughness, infiltration, channel geometry, and rainfall. In that manner, the operator is capable of calibrating the Vflo model to match the observed (at USGS gauges) and simulated hydrographs in regard to the runoff volume, the peak flow, and the time to peak.

3.3.3 HEC-HMS

HEC-HMS (Hydrologic Engineering Center – Hydrologic Modeling System) is a hydrologic software package developed by the Hydrologic Engineering Center (HEC) in 1998. It is the Windows version and successor of the HEC-1 hydrologic software package, which was released by HEC on 1973. HEC-HMS combines empirical and conceptual models to represent the responses of different hydrologic components to hydrometeorological conditions. All the HEC-HMS mathematical components are cause-effect, deterministic formulas.

HEC-HMS is based on lumped modeling, that is, the HEC-HMS watershed is divided to a number of subbasins (depending on the watershed elevation and drainage data) in order to produce what is called a pseudo-distributed model. In a pseudo-distributed model, each subbasin has uniform characteristics inside its boundaries, but creates some measure of spatial variability when it groups with the other subbasins to form the entire watershed. As a part of Tropical Storm Alison Recovery Project (HCOEM, 2003), HEC-HMS hydrologic models were created for all basins in Harris County, using LIDAR elevation data and updated drainage, land use, and soil
information. The Whiteoak HEC-HMS graphical user interface (GUI) is shown in figure 3.7 below:

![HEC-HMS graphical user interface for Whiteoak](image)

Figure 3.7: HEC-HMS graphical user interface for Whiteoak (HCOEM, 2003)

The Whiteoak HEC-HMS models is represented by 41 subbasins. On the other hand, the Whiteoak Vflo distributed model, primarily used in this study, consists of 19,802 cells. Thus, the difference in the ability to depict spatial variations is enormous between HEC-HMS and Vflo models.

Over the years, HEC-1 and HEC-HMS software have been proven capable of simulating successfully rainfall runoff in watersheds and are widely used in hydrologic modeling. The GUI of HEC-HMS allows the user to easily move between the different parts of the program. Precipitation and discharge gage data can be entered either in a simple manual way or in a previously created Data Storage System (DSS) file. The DSS database can be accessed by other software of the HEC family allowing HEC-HMS to
easily exchange information with programs that manage GIS data or calculate water surface profiles for given flow values (i.e. HEC-GeoHMS, HEC-2). The simple way that data are entered, organized, accessed, and visualized in HEC-HMS is an important advantage of HEC-HMS over HEC-1 and other hydrologic models. During this study, the TSARP Whiteoak HEC-HMS model were used for comparison of input and output data at several locations along the watershed. However, because of its lumped character, it lacks the power to take full advantage of the latest detailed elevation and rainfall data sets and meet the objectives of this research.

3.3.4 HEC-RAS

Like HEC-HMS, HEC-RAS (River Analysis System) is the Windows successor of the HEC-2 hydraulic software package, created by the Hydrologic Engineering Center. It performs 1-D hydraulic calculations on natural or constructed channels. HEC-RAS is capable of carrying out hydraulic analysis using either assumptions of steady and unsteady flow. In the steady flow part, it models subcritical, supercritical, and mixed flow regimes. Also, it evaluates the effects of different structures along the channel, such as bridges, culverts, and weirs. Data can be transferred between HEC-RAS and other programs of the same family by utilizing HEC-DSS data files. There are several ways to customize and report the data included in a HEC-RAS model as well as the results of the hydraulic calculations. The operator can create X-Y plots of the river system schematic, cross-sections, water surface profiles, and rating curves. A 3-D view of multiple cross-sections is also provided. Also, the user can select to present his data in a tabular form using predefined tables or customized tables. The HEC-RAS Whiteoak bayou floodway
and Lakeview cross section created under the TSARP project are shown in Figures 3.8 and 3.9 below:

Figure 3.8: HEC-RAS Whiteoak bayou floodway for the 100yr storm (HCOEM, 2003)

Figure 3.9: Lakeview HEC-RAS cross section ((HCOEM, 2003)
A particular issue in this study is the change of Whiteoak 100-yr and 10-yr floodplain elevations due to subsidence and land use development. For this reason, the TSARP HEC-RAS model is used in combination to the Vflo hydrologic results to create scenarios describing the change of Whiteoak watershed during the last 30 years. The Whiteoak HEC-RAS has to be modified incorporating the subsidence information along its cross-sections to account for the continuing subsidence rates. The results of this analysis will be compared with the existing Whiteoak 100-yr and 10-yr floodplain, used in the flood insurance studies.
4. Hydrologic Distributed Model Set Up and Model Calibration

4.1 Introduction

The previous chapter presented issues of data collection and selection of appropriate tools to perform the hydrologic analysis of Whiteoak bayou watershed. This chapter deals with the detailed process of setting up and calibrating the hydrologic model. The model set up phase describes the GIS data preparation and data layers import into Vflo. The calibration and verification processes following the set up step adjust the model parameters to improve fit and test the model accuracy along the watershed. As mentioned before, Vflo accepts spatially varying hydrologic and topographic data sets in the form of grid layers. The different layers should coincide, have the same cell size, and share the same projection. It is important to remember that although Vflo results can be displayed in both metric system and English system units, the topographic input data should only be expressed in metric units. The selection of the cell size depends on the basin size and the spatial variability representative of the dominating hydrologic processes in the study area. The choice of map projection depends on the location of the study area and the properties (shape, area, distance, and direction) to be kept undistorted. No projection can preserve all four properties.

4.2. Choice of Projection

The oldest and most widely used 3D coordinate system is the geographic coordinate system, measured in latitude and longitude units. The latitude lines are parallel, and measure angles between any point and the Equator. That is, north and south
distances from the Equator (Equator is defined as 0 degrees, North pole as 90 degrees north and South pole as 90 degrees south). One degree of Latitude is 69 miles. Lines of constant latitude are called parallels. Longitude lines intersect the poles, run north to south around Earth, and measure angles along the Equator from any arbitrary point. That is east to west distances. Lines of constant longitudes are called meridians. The Prime Meridian that passes Greenwich, London, is accepted to be the 0 degree point of reference.

![Figure 4.1: Latitude and Longitude lines](http://www.maptools.com/UsingLatLon/LatLon.html), August 2005

In order to translate the 3-D shape of Earth on a 2D coordinate system, cartography employs specific functions, which are called projections. For the 2-D representation of the Whiteoak basin, we chose the Universal Transverse Mercator (UTM) projection. The Universal Transverse Mercator is a series of coordinate systems that extend from a latitude of 80° south to 84° north. Starting at −180° of longitude, the globe is divided into 60 zones, each spanning 6 degrees of longitude. Every zone is mapped with a Transverse Mercator cylindrical projection, has a central meridian and two lines of secancy where there is no distortion. Shape is maintained and direction is
preserved locally. Furthermore, inside every zone, there is small area distortion. Figure 4.2 below shows the division of the Earth into zones using the UTM projection.

**UTM Zone Numbers**

Figure 4.2: UTM projection coordinate system. Retrieved from [http://www.colorado.edu/geography/gcraft/notes/mapproj/mapproj_f.html](http://www.colorado.edu/geography/gcraft/notes/mapproj/mapproj_f.html), August 2005

The present study area is located at the UTM15 zone. The UTM projection is commonly expressed in meters. The choice of the specific projection and the selection of the DEM resolution determine the resolution and projection of all other layers that will be used later in the study.

4.2 Watershed Grid

The grid cell size selected for the Whiteoak study is 120m on a side. Whiteoak bayou flows southeast towards downtown Houston and drains approximately 285km²
(110 m$^2$) of watershed. There are four USGS gauges inside Whiteoak boundaries that act as control points for the study. Two of them, #08074500 gauge at Heights and #08074200 gauge at Alabonson, are located along the main channel, while the other two are located on two basic tributaries, Cole Creek and Brickhouse Gully. Figure 4.2 below illustrates the location of the 4 USGS gauges on the projected DEM. The calibration and verification of the model takes place at those 4 points.

Figure 4.3: USGS gauges in Whiteoak watershed

At the final Whiteoak Vflo model, the selected cell size allows the watershed to be represented by 19,802 cells. There is a significant improvement of spatial accuracy of the Vflo model over the HEC Whiteoak models developed during TSARP (www.tsarp.org, 2005). Table 4.1 on the next page compares the number of spatially
homogenous elements that account for the discharge at the 4 USGS gauges in the Vflo Whiteoak model and in the HEC-HMS Whiteoak model.

<table>
<thead>
<tr>
<th>Gauges</th>
<th>HEC-HMS (subbasins)</th>
<th>Vflo (grid cells)</th>
<th>Drainage Area (km²) (from USGS site)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heights</td>
<td>27</td>
<td>15,654</td>
<td>224</td>
</tr>
<tr>
<td>Alabonson</td>
<td>14</td>
<td>6,557</td>
<td>89</td>
</tr>
<tr>
<td>Cole Creek at Deihl</td>
<td>2</td>
<td>1,681</td>
<td>19</td>
</tr>
<tr>
<td>Brickhouse Gully at Costa Rica</td>
<td>3</td>
<td>2,002</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 4.1: Comparison of HEC-HMS and Vflo model ability to explain the spatial variation in the study domain

Clearly, the cell size affects both the running time of the model and the sensitivity of the model to adjustment of its parameters. It is a fact that some spatial information is lost with resampling. In a coarser resolution, the length of the drainage network becomes shorter, and the slope becomes flatter. However, as a first approach, the selected cell size is considered adequate to support the goals of the study and provide efficient times.

4.3 Whiteoak Watershed Delineation

The first step for setting up the model is to derive the flow direction layer of the Whiteoak basin. But what is the extent of the study region? There are established boundaries in Harris County for Whiteoak watershed derived from older elevation models and political reasons. Instead of forcing the LIDAR DEM to comply to the old boundaries, this study accepts the boundaries resulting from the LIDAR DEM processed in ArcView with the support of HECGeoHMS and Spatial Analyst extensions. The
following table 4.2. summarizes 10 basic steps taken in order to prepare the DEM grid for the HECGeoHMS analysis:

<table>
<thead>
<tr>
<th>STEP</th>
<th>ACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Select a DEM study area that overshoots the existing Whiteoak boundaries by at least 1 mile and cut it applying a rectangular graphic. Use the clip grid option under the Transformation menu.</td>
</tr>
<tr>
<td>2</td>
<td>Project the DEM data set from the current projection (State Plane, Texas South Central in feet) to the selected projection (UTM 1983 Zone 15 in meters). Use the grid projection option under the Grid Analyst menu.</td>
</tr>
<tr>
<td>3</td>
<td>Convert the vertical resolution of the new DEM from feet to meters multiplying the grid by 0.3048. Use the map calculator under the Analysis menu</td>
</tr>
<tr>
<td>4</td>
<td>Resample the grid to 120m cell size. Use the resample option under the transform grid menu and select the bilinear method</td>
</tr>
<tr>
<td>5</td>
<td>Examine the river shapefile and compare it with findings from on-site visits at Whiteoak. Remove some historic channels that do not currently exist.</td>
</tr>
<tr>
<td>6</td>
<td>Convert the new river shapefile to grid. Use the convert to grid option under the Theme menu</td>
</tr>
<tr>
<td>7</td>
<td>Divide the river grid by itself to obtain unit value for its cells. Use again the map calculator under the Analysis menu.</td>
</tr>
<tr>
<td>8</td>
<td>Multiply stream unit grid with the DEM grid to let the stream cells get the elevation values. Use map calculator under the Analysis menu</td>
</tr>
<tr>
<td>9</td>
<td>Raise the DEM by a number larger than the maximum grid cell elevation. Use map calculator under the Analysis menu</td>
</tr>
<tr>
<td>10</td>
<td>Merge the stream DEM with the raised DEM. Use the merge option under the Transform Grid menu</td>
</tr>
</tbody>
</table>

Table 4.2: Preparation of data for watershed delineation
Steps 6 to 10 are parts of a procedure called “burning in” streams to a DEM which guarantees that the resulting stream network after the DEM delineation agrees with the original stream network. During the watershed delineation process, the common assumption is that water flows in the direction of the steepest descent. In the burning approach, a deep trench is created in the locations where the stream runs so that the fake extreme slopes adjacent to the stream would lead (and keep) the water in its proper location. Figure 4.4 illustrates the effect of burning a stream shapefile in a DEM:

![Diagram showing the burning process in a DEM](image)

Figure 4.4: Burning the stream shapefile in a DEM

The next step is to process the merged DEM in ArcView with the help of the HECGeoHMS extension. Using Terrain Preprocessing, we follow the standard process that results to the delineation of Whiteoak watershed. The products of the watershed delineation needed for this study are a new Whiteoak watershed boundary, the flow
direction grid and the stream grid. The flow direction grid and the stream grid are the first two files imported into Vflo.

4.4 Derivation of the slope grid layer

The calculation of the slope grid layer is realized with the use of the Spatial Analyst extension. Before that, the DEM of the Whiteoak basin was defined by applying the new boundaries derived from the HECGeoHMS analysis. Figure 4.5 below shows the Whiteoak watershed DEM.

![Digital Elevation Model from LIDAR data](image)

Figure 4.5: Whiteoak watershed DEM

A slope map is computed separately for the basin DEM and the channel DEM using the derive slope option from the Surface menu. Then, the two grid layers of basin and channel slope are merged and converted in ArcView from degree slope to decimal slope, required by Vflo. The slopes along Whiteoak watershed range from 0.001% to 7.9
% (close to the river) with an average value of 0.3% (approximately 18 ft/mi). Figure 4.6 on the next page shows the distribution of slopes in Whiteoak.

![Slope map of Whiteoak watershed](image)

**Figure 4.6: Slope map of Whiteoak watershed**

4.5 Creation of roughness and imperviousness grid layers

As described in Chapter 3, the NLCD Land Use/Land Cover satellite data are used in the study to describe the existing land use conditions in Whiteoak. The land use map is reprojected from the Albers Conic Equal Area projection NAD83 to the UTM15 projection used in the study. Since the resolution of the original data is 30m, the land use map is resampled to a 120m on a side cell size using the nearest neighbor method under the Transform Grid menu. Figure 4.7 on the next page illustrates the land use/land cover map for Whiteoak.
Land Cover Map

Figure 4.7: Land Use/Land Cover map of Whiteoak watershed (representing the period 1992-1995)

Land use variability inside the Whiteoak basin affects two significant parameters of the Vflo model: roughness and percent imperviousness. Roughness is associated with resistance to flow while imperviousness is associated with infiltration and runoff volume. Commonly, hydraulic roughness maps are created from several sources, such as aerial photos or remote sensing of vegetative cover. There are recommended values (or range of values) for hydraulic roughness associated with different land use practices (Vieux, 2004). Table 4.2 on the top of next page summarizes approximate roughness values for certain types of land use:
<table>
<thead>
<tr>
<th>Land Use/Land Cover Classification</th>
<th>Manning ‘n’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential</td>
<td>0.015</td>
</tr>
<tr>
<td>Commercial and Service</td>
<td>0.012</td>
</tr>
<tr>
<td>Industrial</td>
<td>0.012</td>
</tr>
<tr>
<td>Transportation, communications, and utilities</td>
<td>0.015</td>
</tr>
<tr>
<td>Other urban and built up land</td>
<td>0.015</td>
</tr>
<tr>
<td>Cropland and pasture</td>
<td>0.035</td>
</tr>
<tr>
<td>Confined feeding operations</td>
<td>0.05</td>
</tr>
<tr>
<td>Other agricultural land</td>
<td>0.035</td>
</tr>
<tr>
<td>Deciduous forest land</td>
<td>0.1</td>
</tr>
<tr>
<td>Evergreen forest land</td>
<td>0.1</td>
</tr>
<tr>
<td>Mixed forest land</td>
<td>0.1</td>
</tr>
<tr>
<td>Streams and canals</td>
<td>0.03</td>
</tr>
<tr>
<td>Forested wetlands</td>
<td>0.07</td>
</tr>
<tr>
<td>Non-forested wetlands</td>
<td>0.05</td>
</tr>
<tr>
<td>Transitional areas</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Table 4.2: Roughness coefficients for certain types of land use (Source: Vieux, 2004 from Engman, 1986)

In order to create the roughness map for this study, the satellite land cover map is reclassified in GIS using the values from table 4.2. There is inherent error in this process and some engineering judgment for the values used. Some data are lost with resampling. Grid cells belonging under the same general land use category are assigned the exact same roughness factor, even though it might slightly differ from site to site. However, this procedure just aims to approximate spatially, in the best possible way, the location of rough versus smooth land use types.
In a similar manner, the percent imperviousness map is created by reclassifying the satellite land use map. Recommended values for different land use types are given in the Table 4.3 on top of the next page (Bedient, 2002)

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Percent Imperviousness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential</td>
<td>Average 30, Range 22-44</td>
</tr>
<tr>
<td>Commercial</td>
<td>Average 81, Range 52-90</td>
</tr>
<tr>
<td>Industrial</td>
<td>Average 40, Range 11-57</td>
</tr>
<tr>
<td>Institutional</td>
<td>Average 30, Range 17-38</td>
</tr>
<tr>
<td>Open</td>
<td>Average 5, Range 1-14</td>
</tr>
</tbody>
</table>

Table 4.3: Imperviousness by type of land use (Source: Bedient, 2002, from Sullivan et al., 1978)

4.6 Additional watershed information added to the Vflo Whiteoak hydrologic model

The grid layers for the basic Whiteoak model are created and imported to Vflo to represent the hydrologic behavior of Whiteoak. A significant parameter that has not been identified yet, is the channel shape. From the latest version of the HEC-RAS model, channel cross sections downstream of bridges are selected, their location in the watershed is identified from GIS maps and they are manually imported to the corresponding cells of the model. The channel cells between two defined cross sections are assumed to share the shape of the downstream cross section. In total, around one hundred different cross sections define the shape of Whiteoak bayou and tributaries in the model. Vflo allows to import projected shape files belonging to the study domain, and provide markers that improve the orientation and facilitate the visual interpretation of the basin. Figure 4.8 on the next page shows the Whiteoak Vflo Graphical User Interface, including gauges, highways, and cross section marks along the main channel.
Figure 4.8: Vflo Graphical User Interface for Whiteoak watershed; three representative cross sections of the main bayou are highlighted
A scalar bar, the means by which the Vflo model accounts for mathematical approximations and data errors, is shown on the left side of Figure 4.8. The bar supports the model calibration following the OPPA procedure, which is described in the next section.

4.7 Calibration and Verification

The Whiteoak Vflo model includes now all information needed to simulate rainfall-runoff. Before applying it to the analysis of the various scenarios of the study, the model output was calibrated at 4 USGS locations where discharge data were continuously recorded. The calibration process accounts for measurement and random errors, as well as errors caused from resampling. The aim of model calibration is to minimize the difference between the observed and simulated hydrograph in regard to the runoff volume, the peak flow, and the time to peak.

As suggested by Vieux and Moreda (2002), model calibration is a two-step process. First, it tries to balance the runoff volume at the observation points. Then, it seeks to balance the peak flow and the time to peak at the same locations. Vflo Graphic Users Interface provides a scalar calibration bar where multipliers can be used to adjust the magnitude of the significant hydrologic parameters while preserving their spatial pattern. Hydraulic conductivity and rainfall are the main parameters affecting the runoff volume while roughness is a main factor affecting peak flow and time to peak. Since this study already uses gauge calibrated radar rainfall data as an input, and no further rainfall calibration is considered necessary for the simulations.
4.7.1 Choice of Rainfall Events

Two significant rainfall events of the last 4 years were chosen for the calibration and verification of the Whiteoak hydrologic model: November 2003 storm event and October 2002 storm events. The information of the rain gauge calibrated radar rainfall data used in the study is summarized in Table 4.4:

<table>
<thead>
<tr>
<th></th>
<th>Radar Product</th>
<th>Calibration Statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Field Bias</td>
<td>Average Difference (%)</td>
</tr>
<tr>
<td>Nov03</td>
<td>KHGX LII, Tilt 2, 20/55 floor/ceiling, convective Z/R (300,1.4)</td>
<td>0.956</td>
</tr>
<tr>
<td>Oct02</td>
<td>KHGX LII, Tilt 2, 20/55 floor/ceiling, convective Z/R (300,1.4)</td>
<td>1.570</td>
</tr>
<tr>
<td>Sep98</td>
<td>KHGX DHR</td>
<td>1.478</td>
</tr>
</tbody>
</table>

Table 4.4: Calibration information and statistics for the 3 storm events

Section 3.3 of the previous chapter explained briefly the definitions and statistics included in Table 4.4. Figures 4.9 to 4.11 on the next pages show the radar rain total map, exported from Vflo and displayed in ArcView, for each storm event:
Figure 4.9: November 2003 storm event radar rain totals (in.)

Figure 4.10. October 2002 storm event radar rain totals (in.)
Figure 4.11. September 1998 (Frances) storm event radar rain totals (in.)

Tables 4.5 to 4.7 highlight some aspects of the storm events at the 4 USGS locations.

| November 2003 Storm Precipitation, cm (in.) |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| **Gauge**       | **Gauge ID#**   | **Vflo**        | **Vflo**        | **Harris County**|
|                 | Rain - Harris   | Rainfall at     | Average rainfall| gauge value      |
|                 | County          | gauge cell      | upstream gauge  |                  |
| Whiteoak at     | 520             | 9.73 (3.83)     | 15.24 (6)       | 10 (3.94)        |
| Heights         |                 |                 |                 |                  |
| Whiteoak at     | 540             | 18.52 (7.29)    | 14.73 (5.8)     | 17.78 (7)        |
| Alabonson       |                 |                 |                 |                  |
| Cole Creek at   | 590             | 17.65 (6.95)    | 17.58 (6.92)    | 16.99 (6.69)     |
| Deihl           |                 |                 |                 |                  |
| Brickhouse      | 580             | 12.47 (4.91)    | 16.97 (6.68)    | 12.9 (5.08)      |
| Gully at Costa  |                 |                 |                 |                  |
| Rica            |                 |                 |                 |                  |

Table 4.5: November 2003 storm
### October 2002 Storm Precipitation, cm (in.)

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Gauge ID#</th>
<th>Vflo</th>
<th>Vflo</th>
<th>Harris County gauge value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whiteoak at Heights</td>
<td>520/08074500</td>
<td>10.26 (4.04)</td>
<td><strong>15.21</strong> (5.99)</td>
<td><strong>4.39</strong> (1.73)</td>
</tr>
<tr>
<td>Whiteoak at Alabonson</td>
<td>540/08074020</td>
<td><strong>15.06</strong> (5.93)</td>
<td><strong>15.27</strong> (6.01)</td>
<td><strong>14.4</strong> (5.67)</td>
</tr>
<tr>
<td>Cole Creek at Deihl</td>
<td>590/08074150</td>
<td><strong>13.97</strong> (5.5)</td>
<td><strong>17.75</strong> (6.99)</td>
<td><strong>13.89</strong> (5.47)</td>
</tr>
<tr>
<td>Brickhouse Gully at Costa Rica</td>
<td>580/08074250</td>
<td><strong>12.4</strong> (4.88)</td>
<td><strong>14.55</strong> (5.73)</td>
<td><strong>13.79</strong> (5.43)</td>
</tr>
</tbody>
</table>

Table 4.6: October 2002 storm

### September 1998 (Frances) Storm Precipitation, cm (in.)

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Gauge ID#</th>
<th>Vflo</th>
<th>Vflo</th>
<th>Harris County gauge value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whiteoak at Heights</td>
<td>520/08074500</td>
<td><strong>25.5</strong> (10.04)</td>
<td><strong>22.38</strong> (8.81)</td>
<td><strong>25.78</strong> (10.15)</td>
</tr>
<tr>
<td>Whiteoak at Alabonson</td>
<td>540/08074020</td>
<td><strong>21.13</strong> (8.32)</td>
<td><strong>21.44</strong> (8.44)</td>
<td><strong>5.2</strong> (2.05)</td>
</tr>
<tr>
<td>Cole Creek at Deihl</td>
<td>590/08074150</td>
<td><strong>21.02</strong> (8.35)</td>
<td><strong>22.35</strong> (8.8)</td>
<td><strong>26.4</strong> (10.39)</td>
</tr>
<tr>
<td>Brickhouse Gully at Costa Rica</td>
<td>580/08074250</td>
<td><strong>25.58</strong> (10.07)</td>
<td><strong>22.71</strong> (8.94)</td>
<td><strong>13.1</strong> (5.16)</td>
</tr>
</tbody>
</table>

Table 4.7: September 1998 storm
As an example of the current watershed response to rainfall, the observed hydrographs at the 4 USGS gauges are plotted in Figures 4.8 to 4.11 versus the interpolated gauge rainfall intensities for the Nov03 storm. The Time to Peak in each case is calculated from the center of the mass of rainfall to the peak of the hydrograph. Rainfall information for the specific storm is retrieved from the HCOEM web site.

Figure 4.8: Time to peak at Heights during Nov03 storm event
Figure 4.9: Time to peak at Alabonson during Nov03 storm event

![Rainfall Runoff at Cole Creek](image)

1.25 hour

Figure 4.10: Time to peak at Cole Creek during Nov03 storm event

![Rainfall Runoff at Brickhouse Gully](image)

1.75 hours

Figure 4.11: Time to peak at Brickhouse Gully during Nov03 storm event

Generally speaking, three hours of response demonstrated by the larger basins is a very fast time and comparable to the response of other Harris County urbanized watersheds such as Brays Bayou watershed to rainfall. However, these results could also
be influenced by the rainfall distribution and volume in each basin. After setting up the Whiteoak Vflo model, it would be interesting to examine the watershed response under hypothetical spatially homogenous rainfall events of different intensities and durations.

4.7.2 Evaluation of Calibration Results

November 2003 storm is used for the calibration of the Whiteoak Vflo model. Figures 4.12 to 4.15 compare the observed and the simulated discharges resulting after the calibration of the model parameters.

![Discharge comparison at Heights](Figure 4.12: Discharge comparison at Heights)
Figure 4.13: Discharge comparison at Alabonson

Figure 4.14: Discharge comparison at Cole Creek
Figure 4.15: Discharge comparison at Brickhouse Gully

The calibration figures show that the model is capable of matching the peaks and volumes of November 2003 storm at various locations in the watershed. Another way to illustrate whether the model performed well is to plot the simulated versus the observed discharges. Using the same maximum scale values in x and y axes, the points would fall on or cluster around a diagonal line in a good match. Figure 4.16 on the next page includes the plots of the observed versus simulated flows at the 4 locations.
Figure 4.16: Plots of observed versus simulated flow for Nov03 storm event
4.7.3 Evaluation of Verification Results

After calibration, the performance of the Vflo model is verified with a different storm, the October 2002 event. Figures 4.17 to 4.20 compare the observed and the simulated discharges from the October 2002 model runs:

Figure 4.17: Discharge comparison at Heights

Figure 4.18: Discharge comparison at Alabonson
Figure 4.19: Discharge comparison at Cole Creek

Figure 4.20: Discharge comparison at Brickhouse Gully

Figure 4.21 on the next page summarizes the plots of the simulated versus observed discharges for the October 2002 storm:
Figure 4.21: Plots of observed versus simulated flow for Oct02 storm event
4.7.4 Tropical Storm Frances

The model’s performance is also tested with an extreme storm event that occurred on September 1998. Figures 4.22 to 4.25 compare the observed versus the simulated discharges from the T.S. Frances model runs:

![September '98 (T.S. Frances) at Heights](image)

Figure 4.22: Discharge comparison at Heights

![September '98 (T.S. Frances) at Alabonson](image)

Figure 4.23: Discharge Comparison at Alabonson
Figure 4.24: Discharge comparison at Cole Creek

Figure 4.25: Discharge comparison at Brickhouse Gully
It is interesting to notice that the streamflow gauge at Brickhouse possibly failed during the storm simultaneously with the local rain gauge and there are no recordings for approximately 8 hours. This is assumed from various signs. First, the measured discharge stayed almost flat for an extended period of 8 hours. If we go back and check the measured discharges for Brickhouse Gully in the two previously reported storm events, we can see that they reach $200\text{m}^3/\text{sec}$ peaks, that is almost 100% higher peak flows, even though the radar reported rainfall was significantly less. Also, in those storms, the flow rises and recedes fairly fast, as expected to happen in a concrete channel and a fairly urbanized watershed. Based on the above, I conclude that the streamflow gauge was in a failure mode during the event.

Even though in the two smaller basins, the model’s simulations did not provide the best matches for this extreme rainfall event, the model performed in a very satisfactory way at the two larger basins. Figure 4.26 on the next page summarizes the plots of the simulated versus observed discharges for the T.S. Frances, for the three working gauges:
Figure 4.26: Plots of observed versus simulated flow for Tropical Storm Frances

4.7.4. Conclusions

The Vflo Whiteoak model is set up using LIDAR DEM data and satellite land use data and its performance was tested with 3 storm events at 4 locations. In most cases, the model matches the peak flows with less than a 10% error. The $R^2$ values of observed
versus simulated hydrograph vary from 0.65 to 0.98 at the 4 USGS locations, with a median value of 0.9. Some additional statistics calculating error are shown in Table 4.8 below. Cole Creek and Brickhouse Gully subbasins interconnect during the strong storm events creating a shift of rainfall volume that has not been modeled accurately. However, the simulations at the Alabonson gauge, which is the closest gauge downstream the area of our interest are very satisfactory for all three events. Thus, the Vflo hydrologic model of Whiteoak is ready to support the analysis of the various study scenarios that are going to be described in the next chapter.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Formula</th>
<th>Nov03</th>
<th>Oct02</th>
<th>Sep98</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Absolute Error</td>
<td>$MAE = \frac{\sum_{i=1}^{n}</td>
<td>O_{ip} - S_{ip}</td>
<td>}{n}$</td>
<td>7.6</td>
</tr>
<tr>
<td>Mean Absolute Percent Error</td>
<td>$MAPE = \frac{\sum_{i=1}^{n}</td>
<td>O_{ip} - S_{ip}</td>
<td>}{\sum_{i=1}^{n}O_{ip}} \times 100$</td>
<td>3%</td>
</tr>
</tbody>
</table>

Table 4.8: Statistics for Whiteoak watershed model simulations; $O_{ip}$: observed peak flow at the $i^{th}$ gauge; $S_{ip}$: simulated peak flow at the $i^{th}$ gauge; $n$: number of gauges

*calculation used 3 out of 4 gauges
Chapter 5: Results: Part I.
Land Use Analysis

5.1 Introduction

In Chapter 4, a distributed hydrologic model was set up for Whiteoak watershed. The model was calibrated at 4 USGS gauges and exhibited satisfactory performance when tested with three significant storm events of the last 10 years. Furthermore, the Whiteoak Vflo model presented especially good results on the upper part of the watershed that drained in Alabonson gauge. This is particularly important for the fulfillment of the study objectives since most of the analysis included in this chapter focuses on an area called Jersey Village, located approximately 6 km upstream of Alabonson.

The upper part of Whiteoak watershed experienced significant development over the last 30 years that altered its natural drainage pathways. The city of Jersey Village presents a case of additional interest in upper Whiteoak because it is situated close to the center of continuing subsidence resulting from excessive groundwater pumping. The subsidence slowly alters the slopes and eventually may affect the drainage direction of the watershed. Over the last decade, the city of Jersey Village faced increasing flooding occurrences that raised a concern whether a combination of factors related to urbanization adversely affects the rainfall response of the basin. The present research takes advantage of distributed modeling strengths on spatial representation of hydrologic parameters to achieve the following:

a. Represent the spatial variation of roughness and imperviousness over time

b. Evaluate the hydrologic impact of roughness and imperviousness change at specific locations during different time periods
c. Isolate and evaluate the possible hydrologic effect of subsidence at the same locations, during the same time intervals

d. Quantify the combined hydrologic effect of land use change and subsidence in the study area.

The rest of the chapter deals solely with the hydrologic impact of land use change and channel modifications due to development.

5.2 Analysis of the Hydrologic Impact of Land Use Change

In order to evaluate the historical hydrologic impact of development in a watershed, it is necessary to have good knowledge of previous land use conditions. Therefore, historical black and white aerial photos of the upper part of Whiteoak are purchased for the years 1978 and 1987. The photos are georeferenced in ArcMap using street and channel shapefiles and digitized aerial photos from the year 2000 as guidelines. Figures 5.1 and 5.2 on the next page show the upper Whiteoak basin during 1978 and 1987. The photos clearly demonstrate a rapid development of the area between 1978 and 1987. Actually, most of the development that upper Whiteoak experienced during the last 30 years happened during the 80s. The years 1978 and 1987 are specifically selected in this study for comparison reasons: during these years, land subsidence was also measured and recorded in the area by the Harris Galveston Subsidence District.
Figure 5.1: Upper Whiteoak watershed during 1978; the circle emphasizes development

Figure 5.2: Upper Whiteoak during 1987; the circle emphasizes development

The historical aerial photos are further processed in ArcView with the purpose to identify and digitize areas of different development. Because the photos are black and white, it is hard to recognize all the different categories of undeveloped land (in a similar
manner to the satellite data). Thus, the final digitized product includes only 4 categories of land use: residential, low residential, commercial, and undeveloped. These categories are assigned the roughness and imperviousness factors used in Chapter 4. Also, the existing georeferenced color photos of Whiteoak for the year 2000 are digitized and classified similarly in ArcView. Figure 5.3 below shows development of Whiteoak watershed between the years 1978 and 2000.

Upper White Oak Watershed (1978)

(a)

Upper White Oak Watershed (2000)

(b)

Figure 5.3: Urban development in the upper Whiteoak watershed. Panel (a) represents the 1978 land use while Panel (b) represents the 2000 land use.

The evaluation of the land use change effect upstream of Jersey Village (Lakeview gauge) that follows the aerial photo analysis is conducted by importing in the
calibrated Vfio model the digitized layers of roughness and imperviousness for the years 1978, 1987, and 2000. Thus, three variations of the Whiteoak Vfio model are created depending on the stage of development during the specific time frame. Before making the model runs, the effect of overland roughness change was examined in a small test basin.

5.2.1 Vfio Test Basin and the Development of a Revised Whiteoak Vfio Model

A 10x10 grid test basin was created in Vfio to examine simultaneously the effect of overland roughness change and the impact of the model’s roughness calibration factor to the results (Figure 5.4). As mentioned in Chapter 4, Vfio uses scalar multipliers to calibrate the model’s performance. These scalar multipliers range from 0 to 5 and can express a 2 decimal point variation.

Figure 5.4: A 10x10 grid test basin
Table 5.1 below summarizes the results of nine scenarios using the above test basin. Scenarios A1, A2, and A3 examine the downstream impact of overland roughness change from a value of 0.015 (in the residential range) to a value of 0.03 (closer to the undeveloped range for grassland). Scenarios B1, B2, and B3 examine the downstream impact of overland roughness change from a value of 0.03 to a value of 0.045. Finally, scenarios C1, C2, and C3 examine the downstream impact of overland roughness change from a value of 0.03 to a value of 0.06

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Overland Roughness (input)</td>
<td>No Scalar Calibration</td>
<td>Calibration Factor=2</td>
</tr>
<tr>
<td></td>
<td>%</td>
<td>%</td>
<td>%</td>
</tr>
<tr>
<td>A</td>
<td>0.015 to 0.03</td>
<td>26.27</td>
<td>22.78</td>
</tr>
<tr>
<td>B</td>
<td>0.03 to 0.045</td>
<td>13.65</td>
<td>9.69</td>
</tr>
<tr>
<td>C</td>
<td>0.03 to 0.06</td>
<td>22.78</td>
<td>15.98</td>
</tr>
</tbody>
</table>

Table 5.1: Scenarios that evaluate the impact of overland roughness change in basins that use different roughness calibration factors

Table 5.1 shows that the hydrologic impact of land use change on a basin calibrated with a scalar factor of 1.0 (no calibration) is much higher than the impact recorded on a basin that uses a 3.0 scalar calibration factor.

The existing Whiteoak model applies upstream of Alabonson a roughness calibration factor of 2.0 in order to match the observed and the simulated flows. Scalar calibration alters homogenously the values of all the cells in the selected area. However, we would like to keep the commercial and residential roughness values in fixed low levels along the watershed. A reclassification scheme that removes the scalar multipliers, fixes the commercial and residential values at acceptable values and keeps the average roughness value upstream Alabonson at the same level identified in the previous
calibration by altering only the undeveloped roughness value, is described in the following paragraph.

The following steps are taken to update the model in a format that uses no calibration factors:

a. The initial satellite data are reclassified in only 3 categories: Commercial, Residential, and Undeveloped. The Commercial roughness value is kept constant on 0.012. The Residential roughness value is kept constant on 0.02.

b. From ArcView, the number of cells belonging to each class is identified.

c. Using a weighted average analysis, the roughness of the undeveloped area is calculated keeping the average basin roughness constant.

The results from the above calculations suggest an overland roughness value of 0.12 to be assigned to the undeveloped land.

The performance of the revised Whiteoak Vflo model is tested again with same 3 storm events used in Chapter 4. The results of the simulations are very satisfactory and are shown in Figures 5.5 to 5.16. The red line represents the observed hydrograph and the black line, the simulated hydrograph. This modified version of the Whiteoak Vflo model is going to be used for the evaluation of land use change and subsidence scenarios described in the next sections.
Figure 5.5: Observed (red) versus simulated discharge for Nov03 storm event

Figure 5.6: Observed (red) versus simulated discharge for Nov03 storm event
Figure 5.7: Observed (red) versus simulated discharge for Nov03 storm event

Figure 5.8: Observed (red) versus simulated discharge for Nov03 storm event
Figure 5.9: Observed (red) versus simulated discharge for Oct02 storm event

Figure 5.10: Observed (red) versus simulated discharge for Oct02 storm event
Figure 5.11: Observed (red) versus simulated discharge for Oct02 storm event

Figure 5.12: Observed (red) versus simulated discharge for Oct02 storm event
Figure 5.13: Observed (red) versus simulated discharge for T.S. Frances

Figure 5.14: Observed (red) versus simulated discharge for T.S. Frances
Figure 5.15: Observed (red) versus simulated discharge for T.S. Frances

Figure 5.16: Observed (red) versus simulated discharge for T.S. Frances
5.2.2 Evaluation of the Hydrologic Impact of Development Upstream Lakeview

The 100 yr - 24 hr storm event is used for the evaluation of the hydrologic impact of land use change upstream of Lakeview. In this section, the effect of development upstream Lakeview between the years 1978 and 2000 is examined. Figure 5.17 summarizes the model runs for the different land use conditions. The black line represents the model run for the current watershed conditions. The red line simulates the behavior of the watershed under the 1978 land use conditions (without taking into account any channel modifications). Finally, the green line represents the hydrologic effect of both land and channel development.

![Diagram of Land Use change effect at Lakeview](Figure 5.17: Three scenarios analyzing the impact of development upstream Lakeview between the years 1978 and 2000)

The simulation results show that land use change only produced a 6.5% increase in peak flow at Lakeview between 1978 and 2000. However, the combined effect of channel modifications and land use change over the same time period resulted to a 27% peak flow increase. The effect of channel modifications in urbanized areas is dominant.
5.2.3 Hydraulic Analysis of the Land Use Effect

The peak flows resulting from the Vflo model runs at selected upstream Whiteoak cross sections under the different land use conditions are imported in the HEC-RAS model of Whiteoak, prepared under the TSARP project. The model runs showed a significant increase of the 100yr floodplain due to the land use and channel modifications over the last 30 years. Figure 5.18 shows the upper Whiteoak bayou cross sections included in the HEC-RAS model. Figure 5.19 illustrates the increase of the 100yr floodplain due to development.

Figure 5.18: The cross sections of upper Whiteoak Bayou
Figure 5.19: The increase of Whiteoak Bayou 100yr floodplain due to development; the study evaluates only the portion upstream Alabonson
Chapter 6: Results Part II.
Subsidence Analysis

6.1 Introduction

Excessive groundwater pumping supported the needs of a rapidly growing community in Houston and led to widespread subsidence (Zilkoski et al, 2003). In the cases where areas adjacent to coastlines experience subsidence, the hydrologic impact of subsidence on the distressed regions is straightforward: the subsided sections are inundated with water. On the contrary, inland subsidence implications on the hydrologic behavior of a watershed are harder to evaluate and depend on various factors, but mainly the location of the center of subsidence relative to the direction of flow and to the study area. The following analysis takes advantage of the distributed modeling strengths from Vflo in capturing the spatial variability of a watershed to evaluate the local impact of subsidence at Lakeview, Jersey Village, located in the center of the subsidence bowl within Whiteoak Bayou.

6.2 Representation of the 1978 Whiteoak DEM

The Harris Galveston Subsidence District provided the study with the subsidence data and contours from the periods 1978-1987, 1987-1995, and 1995-2000. The contours were then digitized in GIS, transformed into points and interpolated in ArcView using the tension spline interpolation method under the Model Builder menu. The resulting subsidence grid layers shown in Figures 6.1 to 6.3 share the same cell size and same projection with the study DEM.
Figure 6.1: The interpolated subsidence grid for the period 1978 to 1987

Figure 6.2: The interpolated subsidence grid for the period 1987 to 1995
Figure 6.3: The interpolated subsidence grid for the period 1995 to 2000

Figure 6.4 shading illustrates the bowl of total subsidence at the upstream part of Whiteoak watershed. The maximum total subsidence recorded in upper Whiteoak for the period 1978 and 2000 is 1.4 meters. However, it extends over a large area of the basin. In that manner, Lakeview gauge at Jersey Village subsided 1.35 meters in total while Alabonson gauge, located approximately 8.8 km downstream Lakeview, subsided 0.87 meters. That is, Alabonson experienced just half a meter of subsidence less. How this small relative elevation change has affected the basin’s hydrology will be addressed in the analysis in this chapter.

The first task of the subsidence analysis is to recreate the 1978 DEM of Whiteoak, accomplished by removing the interpolated grid layers of subsidence in a backward manner and raising the Whiteoak 2000 DEM to its former elevation. The 1978 DEM is
expected to vary only in slope. The channel network density, location, and geometry have been preserved as they are today. However, as described later in the hydraulic analysis section of this chapter, the cross sections of the upper Whiteoak Bayou have been raised uniformly across their stations to incorporate the change of slope along the main channel.

Figure 6.4: Total subsidence on upper Whiteoak watershed

6.3. Evaluation of the Subsidence Impact

Inland subsidence affects the hydrologic characteristics of a watershed by changing the overland and the channel slope. Changes in the channel slope result to a local increase or decrease of the flow velocity. Changes in the overland slope may, in addition to flow velocity, modify the flow direction and the watershed boundaries. Variations in flow velocities and flow direction influence the peak discharge and the time
to peak of a rainfall event. Watershed boundary modifications affect additionally the rainfall runoff volume.

The first question of interest is how important could a minor change of slope be when added to the mild slopes dominating the area. The Whiteoak land use analysis conducted in Chapter 5 proved that channel related parameters play the most significant role in downstream discharge. In view of that, section 6.3.1 provides a first approach into the evaluation of the subsidence impact in the area with a hypothetical scenario that estimates the hydrologic effect of subsidence due solely to the change of channel slope. Following that, section 6.3.2 evaluates both channel and overland slope change, as well as the possible change of flow direction and area change related to overland slope change. Finally, section 6.3.3 evaluates the hydraulic impact of subsidence on the upper Whiteoak area.

6.3.1 Evaluation of subsidence effect due to channel slope change only

Vflo accepts slope in the form of decimal percent slope, which stands for the percent of vertical difference between the two elevation points of interest divided by their horizontal distance. Figure 6.5.a shows the change of decimal percent slope due to subsidence along the main channel. Seven locations along Whiteoak Bayou are chosen based on the shape of the channel to calculate the average slopes. Point 5 is located at Lakeview gauge while point 7 is located at Alabonson gauge. It is interesting to notice that slope increased during the period 1978 to 2000 by approximately 33% between points 4 and 5 (upstream Lakeview) and by approximately 14% between points 3 and 4. The most upper part of Whiteoak Bayou experienced less than 5% increase in slope. On
the other hand, the channel downstream Lakeview has experienced a decrease in slope by approximately 10% between points 5 and 6 and by less than 3% between the points 6 and 7.

<table>
<thead>
<tr>
<th>Point</th>
<th>2000</th>
<th>1978</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>0.18</td>
<td>0.174</td>
</tr>
<tr>
<td>2-3</td>
<td>0.12</td>
<td>0.115</td>
</tr>
<tr>
<td>3-4</td>
<td>0.075</td>
<td>0.066</td>
</tr>
<tr>
<td>4-5</td>
<td>0.024</td>
<td>0.018</td>
</tr>
<tr>
<td>5-6</td>
<td>0.061</td>
<td>0.068</td>
</tr>
<tr>
<td></td>
<td>0.115</td>
<td>0.118</td>
</tr>
</tbody>
</table>

Figure 6.5.a: Change in the main channel decimal percent slope due to subsidence; point 5 is Lakeview gauge

Figure 6.5.b in the next page and Table 6.1 summarize the slope locations and values calculated for all the tributaries upstream Alabonson. To facilitate the reading of the Table, the tributaries where the slope has increased during the period 1978 to 2000 have been highlighted in green.
Figure 6.5.b: Location of points for analysis

Table 6.1: Change in the tributary % decimal slope due to subsidence; orange numbers are located along the main Whiteoak channel; green values indicate slope increase

<table>
<thead>
<tr>
<th>Interval</th>
<th>Channel Slope (%)</th>
<th>Interval</th>
<th>Channel Slope (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-4</td>
<td>Present: 0.070</td>
<td>1978: 0.054</td>
<td>53-54 Present: 0.348</td>
</tr>
<tr>
<td>1-3</td>
<td>Present: 0.112</td>
<td>1978: 0.106</td>
<td>30-33 Present: 0.124</td>
</tr>
<tr>
<td>5-</td>
<td>Present: 0.295</td>
<td>1978: 0.390</td>
<td>33-35 Present: 0.132</td>
</tr>
<tr>
<td>5-7</td>
<td>Present: 0.086</td>
<td>1978: 0.083</td>
<td>50-34 Present: 0.057</td>
</tr>
<tr>
<td>7-8</td>
<td>Present: 0.117</td>
<td>1978: 0.111</td>
<td>31-33 Present: 0.173</td>
</tr>
<tr>
<td>10-12</td>
<td>Present: 0.191</td>
<td>1978: 0.186</td>
<td>32-55 Present: 0.041</td>
</tr>
<tr>
<td>11-</td>
<td>Present: 0.176</td>
<td>1978: 0.175</td>
<td>36-37 Present: 0.178</td>
</tr>
<tr>
<td>6-9</td>
<td>1978: 0.158</td>
<td>1978: 0.147</td>
<td>56-57 Present: 0.077</td>
</tr>
<tr>
<td>17-52</td>
<td>1978: 0.064</td>
<td>1978: 0.063</td>
<td>40-51 Present: 0.084</td>
</tr>
<tr>
<td>16-15</td>
<td>1978: 0.150</td>
<td>1978: 0.143</td>
<td>41-42 Present: 0.108</td>
</tr>
<tr>
<td>22-24</td>
<td>1978: 0.163</td>
<td>1978: 0.157</td>
<td>51-44 Present: 0.017</td>
</tr>
<tr>
<td>23-25</td>
<td>1978: 0.119</td>
<td>1978: 0.118</td>
<td>43-44 Present: 0.016</td>
</tr>
<tr>
<td>26-27</td>
<td>1978: 0.166</td>
<td>1978: 0.168</td>
<td>45-46 Present: 0.464</td>
</tr>
<tr>
<td>28-29</td>
<td>1978: 0.331</td>
<td>1978: 0.326</td>
<td>44-47 Present: 0.231</td>
</tr>
<tr>
<td>13-14</td>
<td>1978: 0.032</td>
<td>1978: 0.034</td>
<td>39- Present: 0.195</td>
</tr>
<tr>
<td>18-19</td>
<td>1978: 0.276</td>
<td>1978: 0.268</td>
<td>38-58 Present: 0.311</td>
</tr>
<tr>
<td>20-21</td>
<td>1978: 0.208</td>
<td>1978: 0.199</td>
<td>48- Present: 0.219</td>
</tr>
</tbody>
</table>
Two Whiteoak Vflo models were prepared with the 2000 and 1978 channel slopes. The simulated discharges for the 100 year, 24 hour storm event were calculated at the locations 2 through 7 on the main channel. Figures 6.6 to 6.10 show the resulting discharges at points 3 through 7. At point 3, there is already little difference between the 2000 and 1978 simulations. The simulations show that even small changes in channel slope are capable of producing some impact downstream Whiteoak Bayou. Due to the bowl of subsidence, discharge at Lakeview and downstream Lakeview has increased slightly between 1978 and 2000 while at Alabonson it returned at similar levels. In all the analysis of this chapter, the land use changes between 1978 and 2000 have not been accounted.

![Channel Slope Change Impact Only](image)

Figure 6.6: Comparison of model results with and without channel slope change at point 3
Figure 6.7: Comparison of model results with and without channel slope change at point 4

Figure 6.8: Comparison of model results with and without channel slope change at point 5, Lakeview
Figure 6.9: Comparison of model results with and without channel slope change at point 6

Figure 6.10: Comparison of model results with and without channel slope change at point 7, Alabonson
6.3.2 Evaluation of the overall subsidence hydrologic impact on upstream Whiteoak watershed

The previous analysis involved a standard practice in hydrology: the calculation of an average channel slope for the upstream Whiteoak channel network. The purpose of that calculation, compared to the use of grid slopes applied previously in the study, was first to provide an overall idea on how the channel slopes were affected from subsidence. Then, using those average slope values, Vflo simulations were conducted to assess whether a small channel slope change due to inland subsidence would affect the hydrology of locations situated close to the center of subsidence.

However, a calibrated Vflo Whiteoak model, Model A, has already been set up to satisfy the other objectives of the study using grid channel slope and grid overland slope. That same model is used in the analysis of this section to provide us with results representative of the local conditions.

When Model A was created, the DEM of a broader part of Harris County which included the Whiteoak basin was analyzed in HECGeoHMS, in order to derive the Whiteoak watershed boundaries. Now that our boundary is fixed, there is a concern that the new flow direction analysis of the elevated 1978 Whiteoak watershed conducted in HEC-GeoHMS could be influenced not only by the changed elevation but also by the new boundary imposed restrictions. In order to keep all things equal between the 2000 Whiteoak (Model A) flow direction analysis and the 1978 Whiteoak flow direction analysis, the DEM of Model A was re-analyzed in HecGeoHMS. The result of that analysis was Model B, a new 2000 flow direction model for Whiteoak. Then, on a separate analysis, subsidence was removed from the DEM of Model A and the resulting
DEM was processed in HEC-GeoHMS to derive Model C, the 1978 flow direction model. For the creation of Model B (2000) and Model C (1978), the same channel was burned in. All parameters used in the originally calibrated Whiteoak model (Model A) were kept the same. Both Model B (2000) and Model C (2000) had few cells on the far upper part that did not drain downstream. A decision was made not to correct the flow direction in either model, since it was not clear whether the flow adjustments would interfere with subsidence related effects.

After this analysis, the drainage area of the Model B (2000) Whiteoak watershed became a little smaller than the Model A initial basin. However, Model B (2000) compared well with Model A (2000) when it was evaluated at three locations, Jones, Lakeview, and Alabonson (Figure 6.11) using the November 2003 storm event. Figures 6.12 to 6.14 illustrate the comparison results:

Figure 6.11: The location of the three comparison points for the updated and calibrated Whiteoak Vflo model
Figure 6.12: Comparison between the previously calibrated Whiteoak model and the updated Whiteoak model used in the present analysis: Alabonson gauge

Figure 6.13: Comparison between the previously calibrated Whiteoak model and the updated Whiteoak model used in the present analysis: Lakeview gauge
Figure 6.13: Comparison between the previously calibrated Whiteoak model and the updated Whiteoak model used in the present analysis: Jones gauge

Table 6.2 summarizes the number of cells contributing in each of these three locations for the previously 2000 calibrated, the 2000 updated, and the raised 1978 Whiteoak Vflo model.

Table 2: Difference between the number of cells contributing to selected points of the three Whiteoak Vflo models

<table>
<thead>
<tr>
<th></th>
<th>Total number of cells</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Model A</td>
</tr>
<tr>
<td>Jones</td>
<td>1756 (187 channel)</td>
</tr>
<tr>
<td>Lakeview</td>
<td>3417 (363 channel)</td>
</tr>
<tr>
<td>Alabonson</td>
<td>6557 (678 channel)</td>
</tr>
</tbody>
</table>

Using the Vflo Model B (2000), 10 points were selected around Lakeview to examine the possible impact of subsidence on the rainfall runoff discharges. Figure 6.15 shows the location of those points in the Vflo model compared to the subsidence contours. Point 5 is located at Lakeview gauge, in Jersey Village.
Figure 6.15: The Vflo graphical user interface including the 10 points of subsidence analysis; point 5** is located at Lakeview gauge, Jersey Village

Table 6.2 provides the cell type and drainage of the selected points.

<table>
<thead>
<tr>
<th>Point</th>
<th>Cell Type</th>
<th>Drainage (cells/channel cells)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Channel</td>
<td>73 (1 channel cell)</td>
</tr>
<tr>
<td>2</td>
<td>Channel</td>
<td>402 (45 channel cells)</td>
</tr>
<tr>
<td>3</td>
<td>Channel</td>
<td>2169 (249 channel cells)</td>
</tr>
<tr>
<td>4</td>
<td>Overland</td>
<td>8 (0 channel cell)</td>
</tr>
<tr>
<td>5</td>
<td>Channel</td>
<td>3253 (363 channel cells)</td>
</tr>
<tr>
<td>6</td>
<td>Channel</td>
<td>85 (2 channel cells)</td>
</tr>
<tr>
<td>7</td>
<td>Overland</td>
<td>37 (0 channel cells)</td>
</tr>
<tr>
<td>8</td>
<td>Channel</td>
<td>354 (39 channel cells)</td>
</tr>
<tr>
<td>9</td>
<td>Channel</td>
<td>49 (1 channel cells)</td>
</tr>
<tr>
<td>10</td>
<td>Channel</td>
<td>4365 (487 channel cells)</td>
</tr>
</tbody>
</table>
Table 6.3 summarizes the results from the evaluation of the subsidence hydrologic impact on peak discharge.

Table 6.3: Peak discharges at selected channel and overland locations along Whiteoak; the cells that exhibited flow increase due to subsidence are highlighted with green

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.5</td>
<td>11.8</td>
<td>2.54 - decrease</td>
</tr>
<tr>
<td>2</td>
<td>59.1</td>
<td>58.2</td>
<td>2.75 - increase</td>
</tr>
<tr>
<td>3</td>
<td>235.1</td>
<td>243.6</td>
<td>3.49 - decrease</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>2.5</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>357.0</td>
<td>393.5</td>
<td>1.05 - increase</td>
</tr>
<tr>
<td>6</td>
<td>10.9</td>
<td>10.6</td>
<td>0.28 - increase</td>
</tr>
<tr>
<td>7</td>
<td>5.6</td>
<td>5.6</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>35.6</td>
<td>33.5</td>
<td>0.5 - increase</td>
</tr>
<tr>
<td>9</td>
<td>14.2</td>
<td>10.9</td>
<td>2.75 - increase</td>
</tr>
<tr>
<td>10</td>
<td>389.4</td>
<td>393.1</td>
<td>0.94 - decrease</td>
</tr>
</tbody>
</table>

Table 6.3 shows that the overall impact of subsidence on peak flows in upper Whiteoak is minimum. Depending on the location, peak flows slightly increase or decrease. Since the channel plays an important role in flow routing, Table 6.4 is additionally presented summarizing the peak flows from the 2000 and the 1978 Whiteoak models at specific cross sections along the upper Whiteoak bayou. Jones is the furthest upstream cross section while Alabonson is the furthest downstream cross section included in the Table. The numbers confirm once again that subsidence has not significantly contributed to the increase of peak flows at Lakeview. However, it is also observed that there is a tendency of slight peak flow increase at and around Lakeview.
Table 6.4: Peak flows along Whiteoak Bayou

<table>
<thead>
<tr>
<th>Cross Section Name (from upstream to downstream)</th>
<th>2000 Model Peak Flow (cms)</th>
<th>1978 Model Peak Flow (cms)</th>
<th>Peak Flow Change (1978 to 2000) %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jones</td>
<td>183.6</td>
<td>193.7</td>
<td>5.2 -decrease</td>
</tr>
<tr>
<td>Rio Grande</td>
<td>196.4</td>
<td>209</td>
<td>6 -decrease</td>
</tr>
<tr>
<td>West</td>
<td>235.1</td>
<td>243.6</td>
<td>3.5 -decrease</td>
</tr>
<tr>
<td>Tahoe</td>
<td>313.6</td>
<td>311</td>
<td>0.9 -increase</td>
</tr>
<tr>
<td>Louisiana</td>
<td>339.5</td>
<td>333.5</td>
<td>1.8 -decrease</td>
</tr>
<tr>
<td>Windfern</td>
<td>389.4</td>
<td>393.1</td>
<td>0.94 -decrease</td>
</tr>
<tr>
<td>N.Houston-Fairbanks</td>
<td>415.8</td>
<td>419.5</td>
<td>0.98 -decrease</td>
</tr>
<tr>
<td>N.Houston-Rosslyn</td>
<td>437.5</td>
<td>437.7</td>
<td>0.05 -decrease</td>
</tr>
<tr>
<td>Alabonson</td>
<td>440.6</td>
<td>440.6</td>
<td>0</td>
</tr>
</tbody>
</table>

6.3.3. Evaluation of the hydraulic impact of subsidence

The next step of the subsidence analysis is to import the simulated peak flows into the Whiteoak HEC-RAS model created under the TSARP project. The HEC-RAS analysis calculates stage and provides an insight on potential changes of floodplain due to flow variations. This is particularly useful for watersheds with mild slopes, where a small increase or decrease in discharge can result to a noticeable change of the flooded area extent.

The flow changes reported in Table 6.4 are used to support setting up two HEC-RAS models: one representing the present (2000) conditions and another one representing the 1978 conditions. The cross sections of the 1978 model were raised uniformly across their stations to account for the different elevations of 1978. Since no flow data are used downstream Alabonson, it is expected that the results at and below Alabonson contain some level of inaccuracy. A similar statement is valid for the furthest upstream portion of Whiteoak (even though the simulated peak flows of 3 more cross sections upstream Jones were eventually imported in the analysis).
The first task of the HEC-RAS analysis is to evaluate whether the 2000 HEC-RAS model using the peak discharges generated from the updated Vflo model provides similar results to the HEC-RAS model created by TSARP. Figures 6.16 to 6.25 compare the stages generated from the 2000 Vflo/HEC-RAS model and the 2000 TSARP/HEC-RAS model. The blue line with the triangle is the Vflo/HEC-RAS model while the simple blue line is the TSARP model. The comparisons presented very satisfactory results. With the exception of the Rio Grande and West cross sections, the stages of the two models at all other cross sections examined between Jones and Alabonson differed by less than 10 cm (less than one third of a foot).

Figure 6.16: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Jones; 100yr-24 hour storm
Figure 6.17: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Rio Grande; 100yr-24 hour storm

Figure 6.18: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at West; 100yr-24 hour storm
Figure 6.19: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Tahoe; 100yr-24 hour storm

Figure 6.20: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Lakeview; 100yr-24 hour storm
Figure 6.21: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Beltway Eight; 100yr-24 hour storm

Figure 6.22: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Winfern; 100yr-24 hour storm
Figure 6.23: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at N. Houston, Fairbanks; 100yr-24 hour storm

Figure 6.24: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at N. Houston/Rosslyn; 100yr-24 hour storm
Figure 6.25: Comparison of stages between the TSARP HEC-HMS/HEC-RAS and the Vflo/HEC-RAS model at Alabonson; 100yr-24 hour storm

After concluding that the 2000 Vflo/HEC-RAS model provides results representative of the basin, the model is used for the hydraulic evaluation of the subsidence impact. Since the geometric files of the 2000 and 1978 HEC-RAS models are different, it is not feasible to visualize the possible impact of subsidence on the floodplain by plotting both models on the same HEC-RAS geometric graph. Instead, Figure 6.26 plots the top width change at the cross sections of each model. The cross section plots and the tables from the 1978 and 2000 HEC-RAS runs are included in Appendix I and II. The HEC-RAS analysis shows that in all cross sections examined, the 1978 top widths are larger than the 2000 top widths. A larger top width means that the stage is higher and the floodplain is wider. This can be explained by the slightly steeper slopes of 2000 model at the upper part that can result to lower stages.
Figure 6.26: Comparison of cross section top widths between the 1978 and 2000 models along upper Whiteoak Bayou
Chapter 7: Conclusions and Recommendations

7.1. Introduction

This chapter summarizes basic points and conclusions related to the study objectives and provides recommendations for future work. Urbanization in Whiteoak watershed locally altered its topographic and hydrologic parameters through land use change and subsidence. Increased flood occurrences in the upper part of Whiteoak over the last 10 years raised the concern to investigate in depth the interrelation between land use change and subsidence in the region. The focus of the study was an area located in the center of the subsidence bowl. The fulfillment of the research objectives required the integration of GIS data processing techniques and advanced hydrologic modeling to capture the spatial variations and retain the local hydrologic behavior of the watershed.

In addition to the goals of the study, the availability of accurate and complete data sets guided the selection of the modeling approach. Gauge-calibrated radar rainfall data, detailed topographic data derived from LIDAR technology, satellite land use data, aerial photos, on site visits, and channel and street shapefiles supported the Whiteoak model set up. Because of Houston’s susceptibility to heavy rainfalls, a well maintained monitoring rainfall gauge network and a streamflow network were in place and assisted the analysis. A physics-based, fully-distributed hydrologic model, Vflo, processed the GIS data grids and produced rainfall runoff calculations at selected points of interest along the channels and the overland flow cells, for a variety of scenarios.
7.2. Conclusions

7.2.1. Objective 1: To set up a distributed hydrologic model of an urban area

a. The LIDAR DEM was pre-processed in a GIS environment in order to create the flow direction and the channel grid layer required by Vflo. The automated delineation of the watershed in HEC-GeoHMS was conducted without using the District defined drainage boundary and the resulting watershed boundary matched well to the drainage of the Harris County defined basin.

b. The channel configuration should be burned into the GIS DEM before the flow direction analysis. Failing to do so in the Whiteoak study led to a poor match between the actual channel and the GIS derived channel grid on the upper portion of the watershed.

7.2.2. Objective 2: To calibrate and validate the model using significant historical storms of different intensities and durations

a. Three storms events were used to calibrate and validate the model at four locations along Whiteoak watershed (1998-2003 period). From the resulting 12 comparisons between observed and simulated discharges, 9 out of 12 provided very good results. In one case, the streamflow gauge malfunctioned during the event and there were no data for comparison for 8 hours. The other two simulations failed either in capturing the time to peak or the peak flow but provided acceptable results.
b. Accurate representation of the geometry of channel cross sections imported from the latest HEC-RAS Whiteoak model (2004) played a significant role in improving Vflo’s performance.

c. The cell size was selected to be 120 meters on a side. Previous modeling studies of Harris County watersheds, such as Stewart (2001), supported the specific size selection. In the current study, this choice kept the running time of the model reasonable and provided satisfactory simulations in areas sizing from 20 km$^2$ to 220km$^2$.

d. Even though the watershed slopes were mild, the kinematic wave analogy used by Vflo to calculate rainfall runoff worked well.

7.2.3. Objective 3: To focus on a smaller watershed (Lakeview) and to quantify the hydrologic impact of spatially and temporally altered land use patterns

a. The 50% development upstream Lakeview since 1978 was derived by georeferencing and analyzing black and white aerial photos of Whiteoak. Because of the lack of details provided from the black and white color analysis, only three basic categories of land use were used: residential, commercial, and undeveloped.

b. Urbanization increased the peak discharge by approximately 27% at the 100 year level. Only 7% was due to overland land use change. The channel modifications dominated the land use effect (Figure 5.17).
7.2.4 Objective 4: To assess the possible contribution of continuing subsidence to the increased flooding occurrences in the study area

a. Even though upper Whiteoak experienced 1.35 meters of total subsidence over the last years, the change of slope in the watershed was minor since subsidence extended out to a larger area (Figure 6.3). An initial estimation of the impact of the channel slope change at 6 points along the main bayou showed only small, sometimes positive and sometimes negative discharge variations, due to the bowl effect (Figures 6.5, and 6.6-6.10).

b. The overall impact of overland and channel subsidence on upper Whiteoak discharges was estimated at 10 points located in areas of decreasing and increasing slope changes (Figure 6.15). The results again showed minimum varying effects (less than 4%) on peak flows depending on the point location related to the center of subsidence.

c. The difference between the cross section top width of 1978 (no subsidence) and 2000 Whiteoak watershed was evaluated at 10 channel cross sections upstream Alabonson. The 1978 top widths (thus, stages) were found to be slightly larger than the 2000 top widths(Figure 6.26).

7.2.5 Objective 5: To identify the relative effect of land use change and subsidence on the floodplain of the study area

a. The increased flooding occurrences in the upper Whiteoak watershed are related to the upstream land use change during the last 30 years, which significantly expanded the 100 year floodplain (Figure 5.19).
b. Subsidence has not seriously impacted the peak flows of upper Whiteoak. However, it may have affected local ponding incidents, which are critical for the area since most of the houses are built on the street level.

7.3 Recommendations

The following recommendations for future work are proposed based on the achievements of this research work but also on the points left desiring further examination:

- Evaluate the impact of steeper subsidence slopes in the watershed at the 100 year but also at the 10 year level.
- Investigate the steeper subsidence slope model performance under different cell size selection (240 meters and 60 meters).
- Apply the methodology used in this study to calculate the hypothetical subsidence impact of a different shape watershed.
- Expand out to other areas of present concern such as connecting to storm surge models and dealing with coastal flood issues.
LIST OF REFERENCES


TSARP(2002). *Off the Charts*, prepared for FEMA and Harris County Flood Control District.


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<td>Top Width (ft)</td>
<td>2424.81</td>
<td>Top Width (ft)</td>
<td>678.94</td>
<td>190.00</td>
<td>1555.87</td>
</tr>
<tr>
<td>Vel Total (ft/s)</td>
<td>1.87</td>
<td>Avg. Vel. (ft/s)</td>
<td>0.36</td>
<td>3.90</td>
<td>0.47</td>
</tr>
<tr>
<td>Max Chl Dpth (ft)</td>
<td>15.50</td>
<td>Hydr. Dpth (ft)</td>
<td>0.81</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td>Conv. Total (cfs)</td>
<td>287747.4</td>
<td>Conv. (cfs)</td>
<td>8293.0</td>
<td>247666.3</td>
<td>31788.1</td>
</tr>
<tr>
<td>Length Wtd. (ft)</td>
<td>264.56</td>
<td>Wetted Per. (ft)</td>
<td>679.25</td>
<td>195.35</td>
<td>1556.41</td>
</tr>
<tr>
<td>Min Ch El (ft)</td>
<td>96.78</td>
<td>Shear (lb/sq ft)</td>
<td>0.03</td>
<td>0.28</td>
<td>0.04</td>
</tr>
<tr>
<td>Alpha</td>
<td>3.76</td>
<td>Stream Power (lb/ft s)</td>
<td>0.01</td>
<td>1.11</td>
<td>0.02</td>
</tr>
<tr>
<td>Frctn Loss (ft)</td>
<td>0.16</td>
<td>Cum Volume (acre-ft)</td>
<td>3470.79</td>
<td>5413.51</td>
<td>3571.11</td>
</tr>
<tr>
<td>C &amp; E Loss (ft)</td>
<td>0.00</td>
<td>Cum SA (acres)</td>
<td>1580.68</td>
<td>463.88</td>
<td>2041.85</td>
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</tbody>
</table>