Flood Protection Plan

Texas Water Development Board

Brownsville
On the Border By the Sea

March 2006

AMBIOTEC GROUP

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Executive Summary

The purpose of this study was to develop a flood protection plan for the main drainage systems within the City of Brownsville. The frequency and severity of flooding in Brownsville has increased in recent years for three main reasons. First, the region's flat topography combined with regions of relatively high alluvial deposits left over from the migration of the Rio Grande River cause bowl-like features in the topography. These "bowls" fill up with water during large rainfall events and result in large areas of flooding throughout the city. Second, there is an abundance of clay-rich soils throughout the area that inhibits water from being infiltrated. Finally, perhaps the largest issue compounding the flooding problem in Brownsville is the city's recent rapid growth. This type of rapid growth increases the drainage density and further reduces the already minimal soil permeability, resulting in higher runoff peaks and volumes.

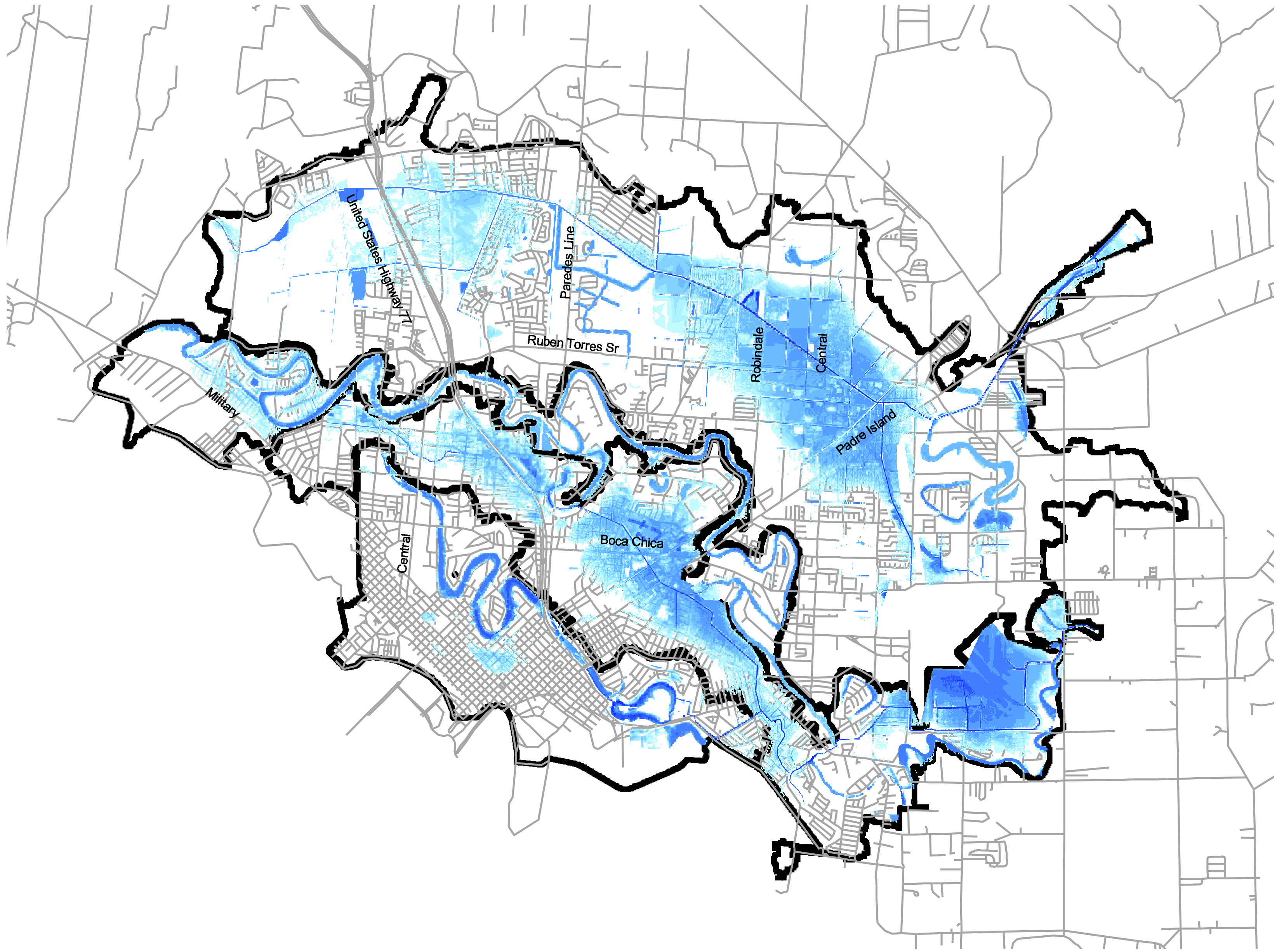
The proposed plan includes both structural and non-structural options. It is designed to reduce the extent and depth of the floodplain within the planning area in a cost-effective manner in addition to preventing a worsening of flooding conditions as development in the area ensues. Among the recommendations of this study is the creation of a regional drainage control agency with taxing authority to focus responsibility, accountability and authority at a single point. Another recommendation includes the development of technically based drainage ordinances to control the unregulated impact of future developments in a cost-effective and consistent manner across the entire watershed system. Such ordinances would restrict the amount of runoff from a site such that it did not exceed the existing, pre-developed flows. They would also state that any future developments could not result in an increase in water surface

elevations anywhere in the watershed. The recommended structural options mainly include the construction of detention ponds, especially multi-use detention ponds. These types of detention areas are desirable because in addition to storing floodwaters during large storm events, they can also be used as a park or recreational area to enhance the beauty of the city. In some areas where detention ponds alone were not adequate to cost-effectively limit flooding, channel modifications including widening and concrete lining of drainage ditches may be recommended.

The planning area for this study encompasses 43.6 square miles and includes four major watersheds: Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1), Resaca de la Guerra (RDLG), North Main Drain (NMD), and Town Resaca (TR). The specific structural and non-structural recommendations that were made for each one of these watersheds is described in subsequent paragraphs as well as in the body of the report.

An analysis of the existing conditions in the study area revealed widespread flooding throughout the area especially in CCDD1 and NMD. In NMD over 30% of the entire land area is inundated with water for the 100-yr storm event. In CCDD1, over 24% of the land area was inundated with water during a 100-yr event. For the two resaca systems, RDLG and TR, a 28% and 13% cover of total land area was observed in each of these two watersheds respectively. The 100-yr floodplains for all four watersheds in the study area may be viewed in Figure ES-1.

Upon completion of the existing conditions analysis, an evaluation of the probable future development scenario was completed as well. This analysis examined the effects that fully developing the study area would have on flow rates and floodplain depths and



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

N

0 1 2 Miles

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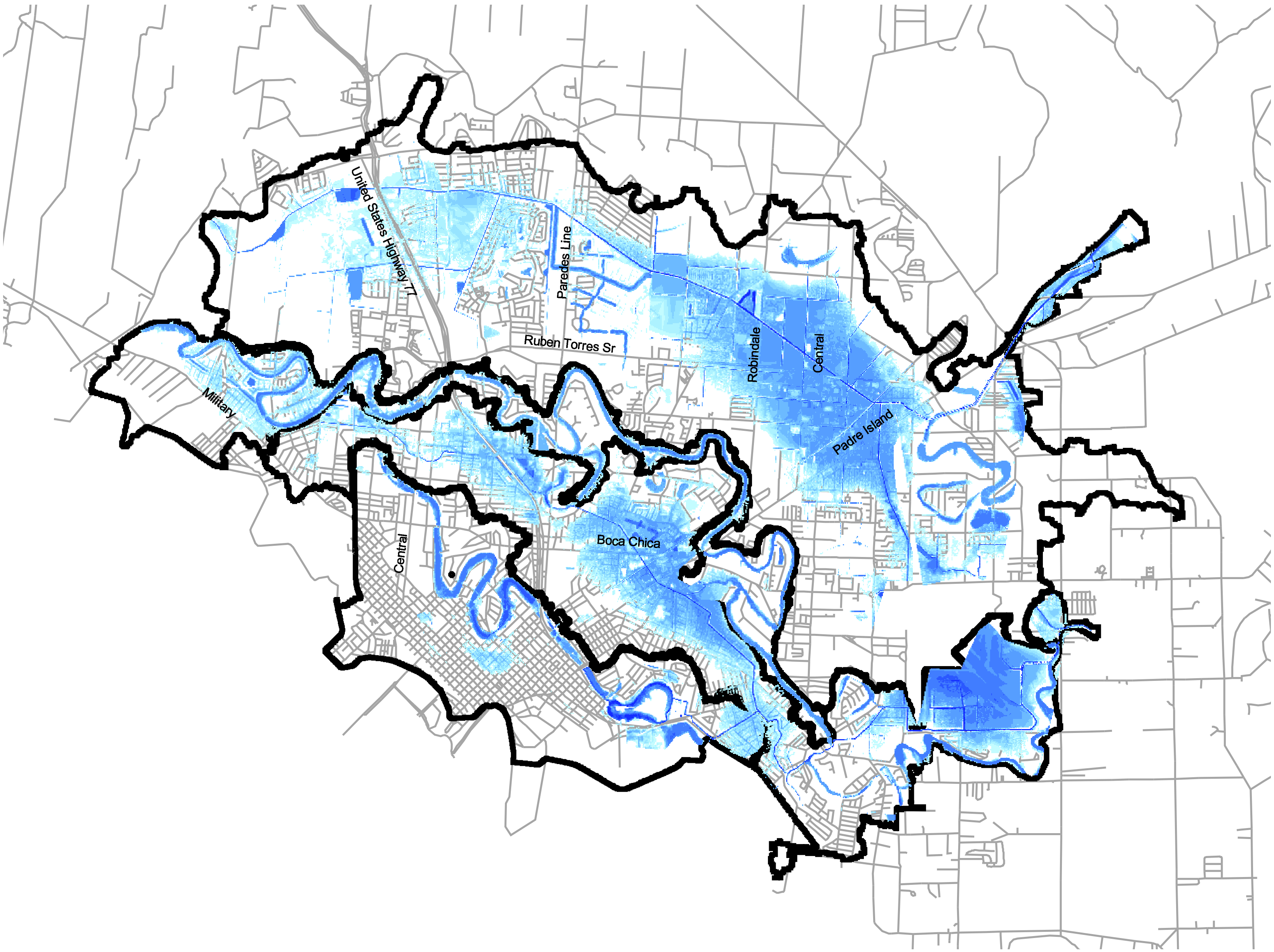
City of Brownsville, TX

Brownsville Watersheds
Existing Development
100-YR Floodplains

Figure ES-1

areas in the drainage ditches and resacas. In NMD the amount of inundated area expanded to nearly 38% of the entire land area and in CCDD1 it expanded to over 29% of the land area. RDLG's total inundated area expanded to over 32% of the watershed area and TR expanded to 17% of the entire watershed area. The 100-yr floodplains for all four watersheds under the full development scenario may be viewed in figure ES-2. The Flood Protection Plan was developed by selecting a number of candidate flood control options; testing the efficacy of each option by running Hydrologic and Hydraulic models to determine the reduction in water surface elevations; estimating the associated flood damages and comparing them to the cost of implementing each option; ranking the alternatives by cost effectiveness; and selecting and giving the highest priorities to those projects that resulted in the greatest flood reduction for a given increment in cost. The study used newly available light detecting and ranging (LIDAR) data, as well as state-of-the-art GIS technologies, to overcome the topographical data quality shortcomings associated with past studies.

The selected projects were organized into a 20-yr CIP consisting of a sequence of four 5-yr CIP plans. The capital cost for the proposed CIP totals over $130 million and includes approximately $33 million in improvements for the North Main Drain, $89 million for CCDD1, and $9 million for Town Resaca. A cost-effective alternative for RDLG was not identified in this portion of the study and thus will be a priority for future work. The majority of the proposed investments are concentrated on the North Main Drain and CCDD1, which by far experience the greatest extent of flooding currently and potentially into the future. Flooding along the resacas is a relatively smaller concern. A



Flood Depth (ft)
- 0 - 0.5
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- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

N

0 1 2 Miles

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Brownsville Watersheds
Full Development
100-YR Floodplains

Figure ES-2

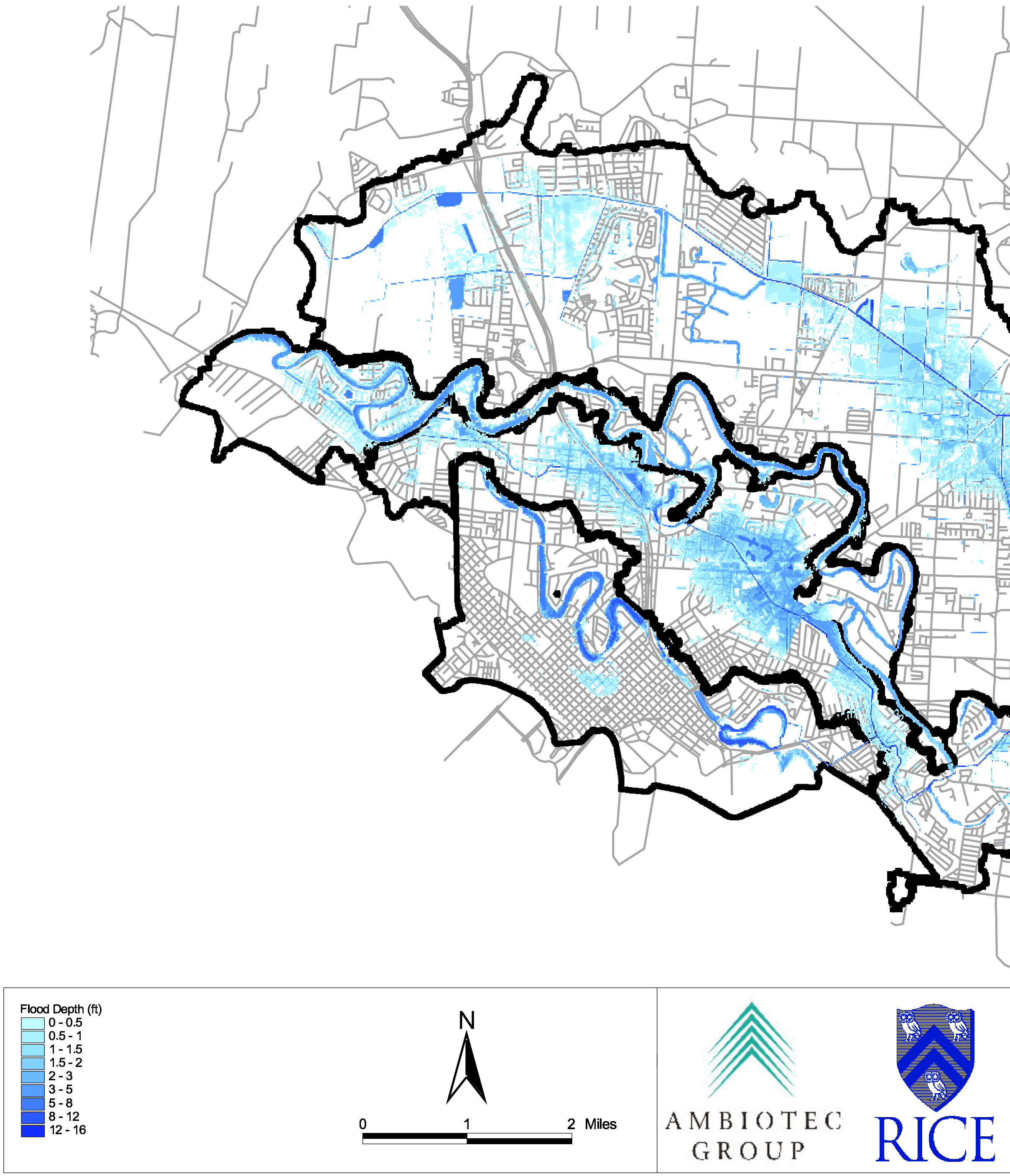
summary breakdown of the timing of the proposed capital investments is presented in the Table ES-1 below.

Table ES-1. Phasing of Proposed Capital Improvement Plan

Watershed	Phase I Capital Costs	Phase II Capital Costs	Phase III Capital Costs	Phase IV Capital Costs	Total Capital Costs	NPV Capital Costs
NMD	$13,890,490	$19,987,500			$33,877,990	$18,917,293
CCDD1	$19,813,650	$19,034,100	$14,014,300	$36,187,470	$89,049,520	$25,442,923
RDLG						
TR	$3,511,750		$5,616,000		$9,127,750	$4,678,092
Total	**$37,215,890**	**$39,021,600**	**$19,630,300**	**$36,187,470**	**$132,055,260**	**$51,191,266**

The proposed plan for the North Main Drain includes the construction of five detention ponds and channel modifications. This results in the reduction of the 100-year floodplain by 22 percent and the out of banks volume by 33 percent compared to the full development "no action" conditions. This reduction corresponds to a drop in computed water surface elevations of over one foot and removes approximately 724 structures from the floodplain relative to the no action scenario. The 100-yr floodplain for NMD under the described plan may be viewed in Figure ES-3.

The structural alternatives proposed for CCDD1 include a culvert improvement at Paredes Line Rd. and the construction of twelve detention ponds as well as the implementation of runoff controls on future development. This reduces the existing development 100-year floodplain by 41 percent and the full development floodplain by 50 percent, respectively; while removing approximately 3985 structures from the floodplain over the no action scenario. The 100-yr floodplain for CCDD1 under the described plan may be viewed in Figure ES-3. The options modeled for Resaca de la



City of Brownsville, TX

Brownsville Watersheds With Implementation of CIP including runoff controls

Figure ES-3

Guerra included the construction of a detention pond near the Quail Hollow area, potential buyout of flood-prone areas near Owen Road, together with dredging of the lower and upper portions of the resaca. However, none of the modeled options proved to be cost-effective. Given this combined with the fact that flooding within this watershed is a fraction of the problem that exists in CCDD1 and NMD, no structural options will be recommended as a result of this portion of the study. It is recommended however that further effort be placed on modeling cost-effective flood mitigation options for this watershed in the future.

The proposed improvements for Town Resaca include dredging of the area near the Gladys Porter Zoo, upgrades to the Impala Pump Station, and lining of the ditch south of the Brownsville PUB Southside waste water treatment plant leading to the pump station. The result of these improvements is a 37% reduction in the 100-yr floodplain and a reduction of the number of structures within the floodplain from 899 to approximately 200. The 100-yr floodplain for TR under the described plan may be viewed in Figure ES-3.

A list of the proposed improvements, grouped by phase, is presented in Tables ES-2 – ES-5. Overall, the plan results in net benefits due to reduced flood damages that exceed $760 million in excess of the net present value of the capital and O&M costs of the proposed improvements over the planning period. The first 5 years of the Phase I CIP is provided in greater detail in Table ES-6. Additional recommendations for the plan implementation include expansion of the hydrologic/hydraulic (H&H) model coverage to include the rapidly developing areas in the Resaca del Rancho Viejo watershed and Cameron County Drainage Ditch #3; integration of the existing and future Resaca del

Rancho Viejo H&H models to allow for the evaluation of inter-watershed diversions which will become an increasingly important element of future cost-effective flood control measures; construction of a rainfall and flow gaging network to improve the predictability of the H&H models; and development of technically-based drainage ordinances to control the unregulated impact of future developments in a cost-effective and consistent manner across the entire watershed system. Potential funding sources are listed as B – Bond Funds and Development or Special District Fees, P-Property Taxes, C – CDBG Grants, F-FEMA, S- Storm Water Utility Fee, and CO – Corps of Engineers Funds.

Table ES-2. Phase I CIP (Years 1-5)

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Construct Price Road Detention Pond	$855,600	B
Construct Old Port Isabel Rd Detention Pond	$5,791,500	B
Construct City Detention Pond Near Owens Road	$2,035,500	C
Construct Coria Detention Pond	$604,200	B
Construct City Detention Pond Near Airport	$4,036,500	B
Construct levee around southern portion of Airport	$567,190	B
Total NMD:	**$13,890,490**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Funding
Implement Technically Based Runoff Controls for New Developments	---------	S
Remove and Replace Weir Structure @ Paredes Line Road	$179,400	B
Install side weir at Exst. Super Walmart Detention Pond	$229,500	B
Install side weir at Exst. UP Railroad Detention Pond on Nopalitos Drain	$148,500	B
Purchase Land for Dana Road Detention Ponds	$1,402,500	B
Purchase Land for Robindale Road Detention Pond	$874,500	B
Purchase Land for FM 802 Detention Pond	$2,805,000	B
Construct Towne North Detention Pond	$7,863,000	B
Brownsville Golf Center Detention Pond - 180 Acre Feet	$5,882,250	B
Purchase Land for Minnesota & Austin Road Detention Pond	$429,000	B
Total CCDD1:	**$19,813,650**	

Town Resaca		
Proposed Improvement	Estimated Costs	Funding
Property Buyouts	$750,000	F
Impala Pump Station Upgrade	$828,000	B
Line Ditch from South WWTP to Impala Pump Station	$1,933,750	C
Total TR:	**$3,511,750**	
Total Costs:	**$37,215,890**	

Table ES-3. Phase II CIP (Years 6-10)

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Line ditch to top of bank from 77/83 to Airport	$19,987,500	B
Total NMD:	**$19,987,500**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Detention Pond upstream of UP Railroad - Nopalitos Drain	$5,620,500	B
Construct Detention Pond on Minnesota and Austin Road	$3,684,600	B
Construct Dana Road Detention Ponds (2)	$9,729,000	B
Total CCDD1:	**$19,034,100**	
Total Costs:	**$39,021,600**	

Table ES-4. Phase III CIP (Years 11 – 15)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Detention Pond upstream of UP Railroad - Ditch No. 1	$5,620,500	B
Construct Public Works Detention Ponds (2)	$2,975,250	B
Construct Detention Pond on Minnesota near Airport	$5,297,500	B
Complete CCDD1 Detention Pond	$1,596,000	B
Construct Robindale Road Detention Pond	$7,120,800	B
Total CCDD1:	**$22,610,050**	

Town Resaca		
Proposed Improvement	Estimated Costs	Possible Funding Source
Dredge Town Resaca near Brownsville Zoo	$5,616,000	CO
Total TR:	**$5,616,000**	
Total Costs:	**$28,226,050**	

Table ES-5. Phase IV CIP (Years 16 – 20)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Construct FM 802 Detention Ponds (2)	$20,320,500	B
Replace FM 3248 (Alton Gloor) culvert with bridge structure	$942,540	B
Replace Paredes Line Rd. culvert with bridge structure	$928,740	B
Replace Dana Road culvert with bridge structure	$942,540	B
Replace Old Port Isabel Rd. culvert with bridge structure	$403,650	B
Replace FM 802 culvert with bridge structure	$942,540	B
Replace International Blvd. culvert with bridge structure	$1,106,760	B
Replace RR culvert near 48 with bridge structure	$2,004,450	B
Total Costs:	**$27,591,720**	

Table ES-6. Detailed Breakdown of Phase I CIP

Year 1 of Phase I CIP	
Improvement	**Capital Cost**
Organizational restructuring/ordinances/financing structure	N/A
Plan, design, and construct Towne North Detention Pond	$8,042,400
Total	$8,042,400
Year 2 of Phase I CIP	
Improvement	**Capital Cost**
Plan, design, and construct Brownsville Country Club Detention Pond	$5,882,250
Construct City Detention Pond Near Owen's Road	$2,035,500
Purchase land for Dana Road Detention Pond	$1,402,500
Total	$9,320,250
Year 3 of Phase I CIP	
Improvement	**Capital Cost**
Plan, design, and construct City Detention Pond Near Airport	$4,036,500
Impala Pump Station Upgrade	$828,000
Purchase land for Robindale Rd. Detention Pond	$874,500
Install side weir at Exst. Super Walmart Detention Pond	$229,500
Price Rd. Detention Pond	$855,600
Total	$6,824,100
Year 4 of Phase I CIP	
Improvement	**Capital Cost**
Construct levee around southern portion of Airport	$567,190
Purchase land for FM 802 Detention Pond	$2,805,000
Construct Old Port Isabel Rd. Detention Pond	$5,791,500
Total	$9,163,690
Year 5 of Phase I CIP	
Improvement	**Capital Cost**
Construct Coria Detention Pond	$604,200
Install side weir at Exst. UP Railroad Detention Pond on Nopalitos Drain	$148,500
Purchase land for Minnesota & Austin Road Detention Pond	$429,000
Property buyouts TR	$750,000
Line Ditch from South WWTP to Impala Pump Station	$1,933,750
Total	$3,865,450
Total for Years 1-5	**$37,215,890**

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1.0 INTRODUCTION

Flooding is now the most frequent and costly of all natural disasters in the United States (NWF, 1998). During the last decade alone, damages from floods have exceeded over $4 billion in the United States. These heavy losses have focused a greater need on flood control. Traditionally, flood control methods have focused on the structural alterations of channels as well as the building of dams and reservoirs. More recently, flood management has shifted towards a more effective and environmentally sound approach to flood control. This especially holds true in urbanizing areas where a significant increase in flooding has been observed over the past several decades (Benavides, 2002). The more modern approach to minimizing flood damages centers around the principle of floodplain management.

Floodplain management uses a combination of both structural and nonstructural techniques to minimize flood damages. Structural methods include such options as channel modifications, detention/retention areas, diversions and levees; while nonstructural methods involve buyouts of flood prone areas, development and land use controls, flood proofing, flood warning systems, drainage maintenance programs, and public awareness/information programs (Benavides, 2002).

While floodplain management techniques were not as widely used historically, they are not new concepts. A prime example of the use of floodplain management techniques occurred during the 1970s at The Woodlands, Texas, located just north of Houston. A central part of the floodplain management strategy at the Woodlands was the integration of detention ponds that served multiple purposes including flood control, aesthetic, as well as recreational and environmental benefits. (Bedient et al., 1985). A key element of this strategy is to control runoff from developing areas before it gets to the main channels, where control becomes more difficult and expensive.

1.1 Background

Rapid development in Brownsville, Texas has caused an increase in the magnitude and frequency of flood events. The region's flat slopes and clay-rich soils make the region especially susceptible to flooding. The problem is further compounded by the area's rapid development. Generally, rapid development increases the drainage density and reduces the soil permeability. This leads to higher runoff volumes and peak flows that further stress the carrying capacity of the primary drainage structures. In the case of Brownsville, development has caused enough additional stress on the City's two major drainage ditches that flooding is now a regular and more severe occurrence. Furthermore, because the City has not reached full development, and growth continues at an accelerated rate, this problem will only get worse into the future.

1.2 Overview of Flooding Problem

There are two main man-made drainage ditches in Brownsville. The first ditch is located at the northernmost portion of the city and is called Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1). The second ditch, North Main Drain (NMD), runs through the center of the city between Resaca de la Guerra (RDLG) and Town Resaca (TR) before draining into the Brownsville ship channel. The inadequacy of these two ditches to drain excess runoff during storm events is the main source of flooding in Brownsville. Historically, flooding in the resacas has been minimal to non-existent.

Generally, most of the flooding problems were located in the area around the ditches and were limited to severe street flooding with some associated structural damage. However, it is important to note that as the area becomes more developed and the flooding problem is worsened, there exists more potential for increased structural damage in larger areas of the City.

The Flood Insurance Rate Map (FIRM) for the City of Brownsville reflects the flooding that occurred in the 1967 event from Hurricane Beulah. In addition to the out-of-bank flooding of the primary drainage system, these maps show that there are other flood-prone areas located away from the main channels, that are the result of an inadequate secondary drainage system (Rust, Lichliter, and Jameson, 1996). While this type of flooding is still a concern, it is a lower priority than the out-of-bank flooding of the primary drainage system since secondary flooding cannot be mitigated without first addressing the inadequacies of the primary drainage system.

Other major storm events occurred in 1967, 1984, 1996, and 1997 and caused extensive damage in many regions throughout the City and illustrate the need for the city to develop cost-effective solutions to the regions flooding problem especially as the area becomes more developed.

The 1967 event was the highest daily rainfall for the period of 1896 to 1991 with 12.1 inches. This exceeds the 100-yr rainfall of 11.7 inches in 24 hours for Brownsville. For this event, two daily totals were greater than 10 inches, two were between 7 and 8 inches, two daily totals were between 6 and 7 inches, and 11 daily totals were between 5 and 6 inches. The event in October 1996, which dropped 10.6 inches in 24 hours, was the second highest daily historical rainfall (Rust, Lichliter, and Jameson, 1996). Both events caused significant street flooding and damage to some structures. Other significant rainfall events for the City of Brownsville may be viewed in Table 1.1.

Table 1.1. Historical Rainfall from the National Weather Service Rainfall Gauge at the Brownsville Airport

Date	Rainfall Total (in)	Remarks
Sep-67	15.4	Hurricane Beulah
Aug-80	6.9	Hurricane Allen
Sep-84	15.2	
Sep-88	5.4	Hurricane Gilbert
Oct-96	10.6	Tropical Storm Josephine
May-04	8.5	

1.3 Project Scope

This purpose of this project is to develop structural and non-structural options to reduce the extent and depth of the flooding in the primary drainage system of the City of Brownsville. The study used newly available light detecting and ranging (LIDAR) data, as well as state-of-the-art GIS technologies, to overcome the shortcomings with regard to detailed topographical data of past studies. This newly available data provides a distinct advantage for floodplain modeling over past efforts.

The following objectives were addressed in this study:

1) Model the runoff response of the North Main Drain and Cameron County Drainage District No. 1 Ditch No.1 using a lumped parameter hydrological model, HEC-HMS,

with the aid of LIDAR derived topographic data and both NEXRAD and design storm rainfall data.

2) Model the runoff response of two resaca networks, Resaca de la Guerra and Town Resaca, using a physics-based distributed model, Vflo™ also with the aid of LIDAR derived topographic data and both NEXRAD and design storm rainfall data.
3) Use the lumped and distributed models and a hydraulic model, HEC-RAS, to compute water surface elevations throughout the watershed and determine the extent of the flooding problem within the drainage ditches and resacas through floodplain delineation.
4) Determine the economic impact of the delineated floodplains on local structures in the area.
5) Develop a cost-effective Capital Improvement Plan (CIP) consisting of both structural and non-structural flood mitigation strategies to reduce economic losses resulting from flood events in a cost-effective manner.

1.4 Previous Flood Studies

Several studies have been completed in the Brownsville area over the past three decades. In 1976, Balli & Associates in association with Henningson, Durham and Richardson, completed an urban waterways study that was focused on the resaca networks. The goal of the study was to develop a long-range master plan for restoring, preserving, and utilizing the resacas to the maximum of their potential. While the study focused on issues of sedimentation, water quality, and resaca use, the study did point out the most flood-prone areas in the region. They noted that the flood-prone areas were not along the resacas but in low areas between the resacas and in the areas between the Rio Grande River and the secondary levee. However, while they claimed that flooding along the resacas was unlikely, they acknowledged that Resaca flooding was possible if one or more of the culverts connecting the resaca pools together became obstructed. They concluded that regular maintenance of the hydraulic structures in the resacas was essential (Balli & Associates and Henningson, Durham and Richardson, 1976).

On September 27, 1985 the City of Brownsville authorized Hogan and Rasor, Inc., in association with Mejia, Hampton and Rose, Inc. and R.J. Brandes Co., to investigate the flooding of properties along the major drainage facilities comprised of resacas, ditches, and channels. The project was prompted as a result of the increased flooding that had been experienced in Brownsville over recent years and the realization that these problems would likely be compounded in the future as development within the watersheds continued. The purpose of the study was: 1) to understand how the drainage facilities function hydraulically under existing conditions; 2) to identify where the major existing flooding problems occur; and, 3) to provide a sound technical basis for proceeding with more detailed analyses and planning efforts that will result in effective solutions to existing and future flooding problems (Hogan & Rasor, Inc., 1986).

The study presented a number of conclusions about the nature of flooding in Brownsville. First, that flooding problems along the North Main Drain (NMD) are generally due to the insufficient water-carrying capacity of the watershed due to undersized structures, small channel cross sections or poor channel alignment. Second, that the effects of flooding in the Cameron County Drainage District No. 1 Ditch No.1 (CCDD1) are minimal because the area is agricultural in character; but that as urbanization occurs in the future, the amount of water carried by the ditch will increase. As a result, CCDD1 presents an opportunity to properly manage and plan the growth so as to minimize future problems. And third, that Town Resaca (TR), experiences a greater increase in water surface elevations throughout its length during large storm events than Resaca de la Guerra (RDLG). This was attributed to the higher percent of urbanization in the watershed; more flow restrictions in the channel, and a difference in overbank storage characteristics. Therefore, as the less urbanized areas of RDLG become more developed, it is likely that flooding problems in this watershed will worsen.

Specific recommendations for measures to mitigate flooding problems were presented in the Master Drainage Plan for the City of Brownsville that was drafted in August of 1987 (Hogan & Rasor, Inc., 1987). The goal of the recommendations were to provide flood protection for the 100-yr storm in the major watersheds centering on the major drainage facilities comprised of resacas, ditches and channels. The major drainage facilities were analyzed using HEC-1 and it was determined that the severe flooding problems in the area were a result of the natural flatness of the Lower Rio Grande Valley, the large amounts of rainfall, and the development of many flood-prone areas without installation of proper drainage facilities. The proposed plan for immediate improvements in the area included a combination of structural improvements, increased detention storage, and routine maintenance of channels and hydraulic structures.

Hogan and Rasor (1986) site two major issues that limit the accuracy of their study. The first major limiting factor is the lack of detailed topographic information and ground elevation data within the watersheds. At the time of the study, the only published topographic maps in the area were U.S. Geological Survey (USGS) quadrangle sheets with five-foot contour lines. They discovered that this level of detail was not adequate to accurately describe the available storage volume within each drainage ditch and resaca pool. Nevertheless, because this was the only published topographic data available at that time, the USGS quadrangle sheets were used in conjunction with aerial photographs for that particular study.

The second major limiting factor of this study was the unavailability of detailed field data that described the elevations and dimensions of outlet structures and roadway crossings. In addition, channel cross-section data for the main drainage ditch (NMD) that discharges into the Brownsville Ship Channel was not available for this study. This data is necessary to correctly simulate the overall hydraulics. As a result of these two major shortcomings, Hogan and Rasor point out the importance of satisfying these data deficiencies in subsequent studies.

A follow-up to the 1987 Master Drainage Plan was prepared by Rust, Lichliter and Jameson in 1996. The study aimed to: 1) identify the causes of flooding; 2) develop a plan for the orderly implementation of cost-effective solutions to the flooding plan; 3) eliminate flooding conditions, resulting flood damages, safety and access problems and health hazards, and 4) develop a plan for the future anticipated growth of Brownsville to insure properly controlled drainage. The study concluded that the NMD and the CCDD1 do not have the capacity to convey even the five-year frequency run-off event without causing significant out-of-bank flooding. Stormwater from heavy rainfall events near the resacas was contained within the banks and any areas of localized flooding were due to an inadequate secondary drainage system. However, a detailed analysis of the secondary drainage system was not examined in this report. The study recommended a short-term flood control Capital Improvement Plan (CIP) consisting of structural improvements at a few isolated locations, and the pumping down of water surface elevations prior to a large storm. Furthermore, the study addresses the idea of dredging the resacas to create additional storage for stormwater. While dredging could potentially enhance storage, water quality and environmental habitat, it should be noted that dredging alone will not result in additional storage unless the permanent pool levels of the resacas are also lowered.

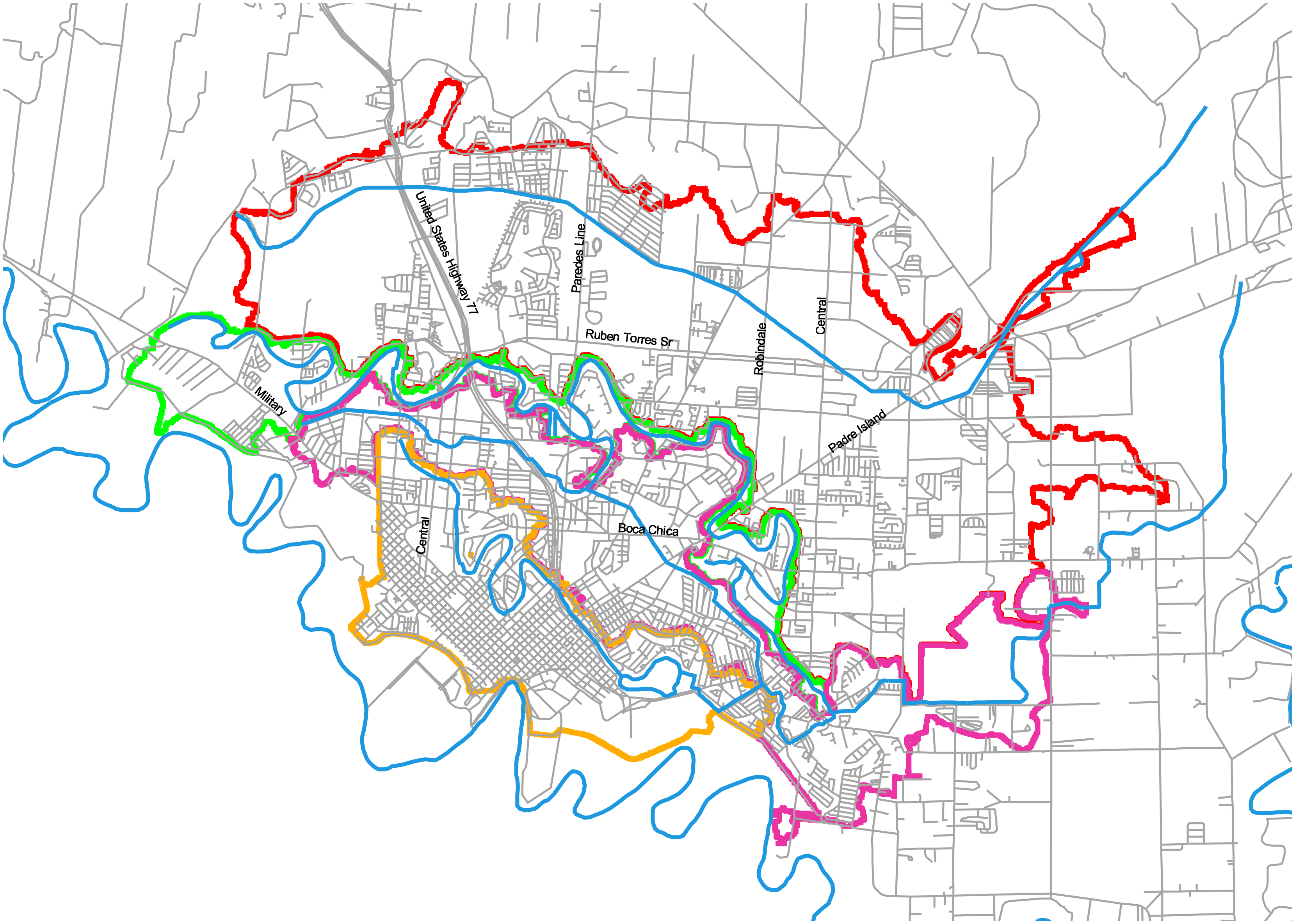
2.0 DATA COLLECTION

2.1 Study Planning Area

The study planning area encompasses 43.6 square miles and includes four major watersheds: Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1), Resaca de la Guerra (RDLG), North Main Drain (NMD), and Town Resaca (TR) (Figure 2-1). While the watershed boundaries are defined topographically, the general boundaries of the study area include the Rio Grande River to the South, Indiana Ave. to the east, and Alton Gloor to the north and west. LIDAR derived topographic data reveals that ground elevations in the study area range from approximately 40 feet mean sea level (msl) at the western edge of the area to below 5 ft msl on the eastern side near the Brownsville Ship Channel.

The study area contains numerous drainage ditches, bridges, drainage structures, culverts and pumping stations. Information about these features was collected from a variety of sources including field surveys and published reports. Photographs and field measurements were taken at all accessible hydraulic structures. Data from previous reports were used to characterize inaccessible hydraulic structures. A comprehensive list of the various hydraulic structures in each watershed may be viewed in Tables 4-4 and 4-5 for NMD and CCDD1 respectively, and Tables 5-1 and 5-2 for the two resaca networks.

The topographic model for this study was derived from LIDAR data collected in 2001 for the City of Brownsville and provides a comprehensive set of elevation points for every 10-ft by 10-ft area. An image of this data set may be viewed in Figure 2-2.



Major Streams
Streets
Watersheds
CCDD1
NMD
RDLG
TR

N

0 1 2 Miles

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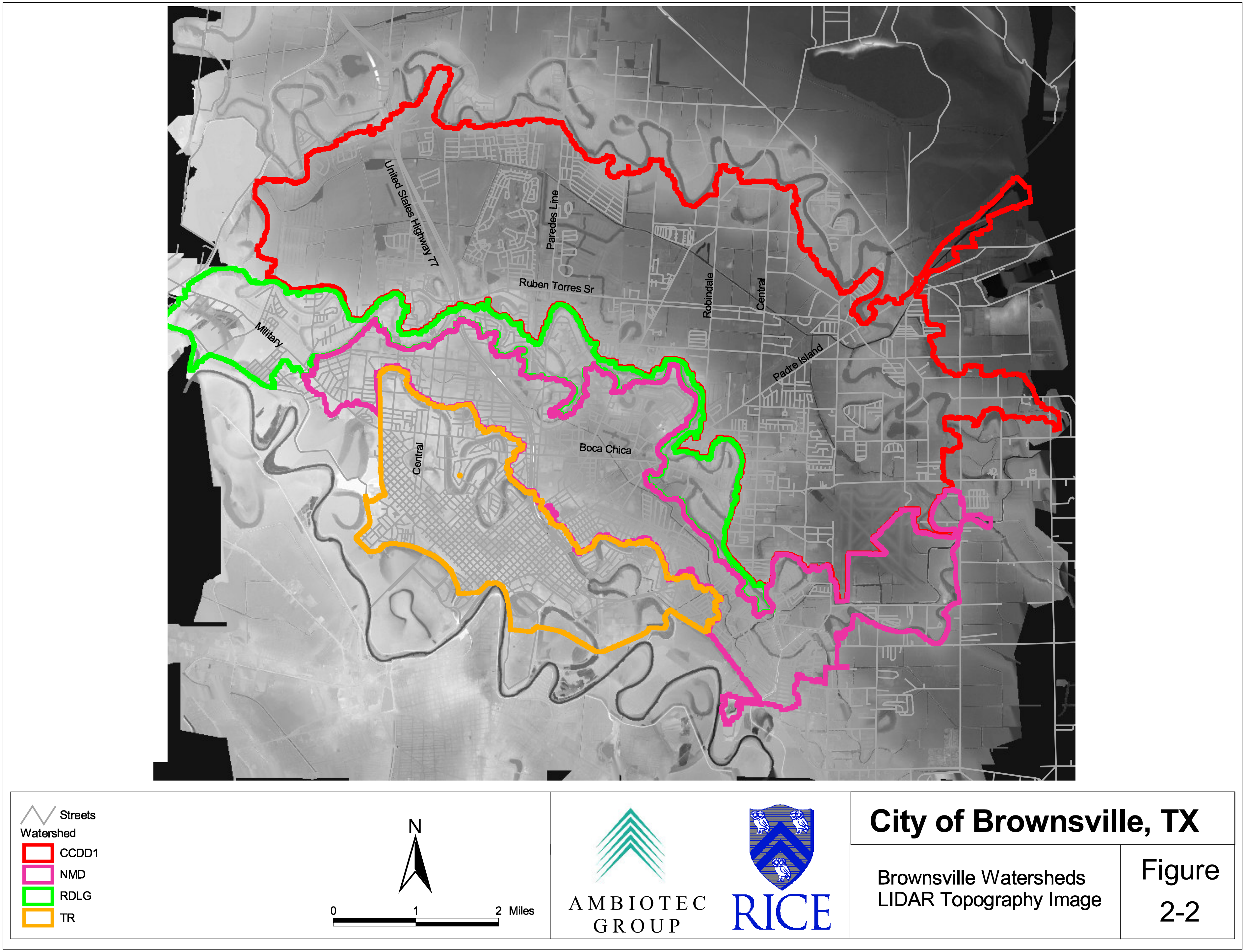
City of Brownsville, TX

Study Area

Figure 2-1



City of Brownsville, TX

Brownsville Watersheds
LIDAR Topography Image

Figure 2-2

Resaca sediment survey data recently collected by the US Corps of Engineers was used to correct the LIDAR resaca bottom elevations. Land use maps were created through the interpretation and field verification of aerial photos also collected in 2001 as discussed in section 2.4. In addition to this land use data, the Public Utilities Board provided information regarding the number and location of residential and commercial structures in each watershed. This permitted the estimation of the number of structures impacted by the delineated floodplains in each watershed.

2.2 Existing Drainage System

The drainage system for The City of Brownsville has two major components: The primary drainage system, and the secondary drainage system. These two systems are typical of any urban storm drainage system across the United States. The primary drainage system for The City of Brownsville consists of two major drainage ditches, specifically North Main Drain (NMD) and Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1), and two resacas, Town Resaca (TR) and Resaca de la Guerra (RDLG). The secondary drainage system is made up of a network of open and closed conduits that convey storm runoff from frequent, low-intensity storms to the primary system. This system is complemented in urban areas by a street system graded to convey sheet flow runoff when the capacity of the secondary system is exceeded during larger storm events. In some areas, the street grading is inadequate to transport this excess flow and extended periods of street ponding and possible structural flooding results.

2.2.1 Primary Drainage System

The primary drainage system in Brownsville, Texas consists of two major man-made drainage ditches and two resacas (Figure 2.2). A third Resaca, Resaca del Rancho Viejo, lies in the northern portion of the Brownsville area but was not included in the scope of this study.

Resacas are isolated ox-bow lakes, created by changes in the Rio Grande's course. Over time, overbank flooding of the Rio Grande caused deposition of sediment along the banks of the channel causing the resacas' characteristic high banks. Today, resacas are characterized as a series of shallow connected ponds with constant pool water levels. The water levels are controlled by weir structures built within and between the various ponds. Brownsville resacas have become attractive amenities and prime real estate property and thus are no longer seen as just drainage channels but an integral part of the community.

The two man-made drainage structures are the Cameron County Drainage District No.1 Ditch No. 1(CCDD1) and the North Main Drain (NMD). These ditches were constructed between the high banks of the resaca systems to drain the stormwater runoff that could not reach the resacas. Both ditches are trapezoidal in shape with portions having concrete side slopes to prevent erosion and convey water more effectively. The drainage areas of these four water bodies (CCDD1, RDLG, NMD, and TR) make up the study area. A brief discussion of each of the four watersheds is discussed below.

Cameron County Drainage District No.1 Ditch No. 1

The CCDD1 watershed (Figure 2-1) comprises the northernmost portion of the study area. The entire watershed is approximately 23 square miles and has average slopes that range between 0.1% to 1.2%. The ditch is approximately 11.5 miles long and generally runs in a southeasterly direction for the majority of its length before making a sharp turn to the northeast where it drains into San Martin Lake. This watershed is approximately 33% undeveloped; it consists of mixed residential, commercial and agricultural land uses and is experiencing very rapid urban growth.

Resaca de la Guerra

The RDLG watershed as viewed in Figure 2-1 is located between the NMD and CCDD1 watersheds. The resaca traverses in a northwest to southeast direction for approximately 17.3 river miles and outfalls into NMD through a weir structure. The watershed is approximately 4.6 square miles in area with slopes averaging between 0.6% to 1.7%. This watershed is approximately 25% undeveloped; and consists of predominantly high intensity residential land use.

North Main Drain

Located just south of RDLG is the NMD watershed (Figure 2-1). NMD runs west to east for 16.6 river miles through the most heavily urbanized areas in Brownsville before it outfalls into the Brownsville Ship Channel. The watershed is roughly 9.6 square miles with slopes averaging between 0.1% to 1.0%. This watershed is almost completely developed, and consists primarily of high intensity residential and commercial/industrial land uses.

Town Resaca

The TR watershed (Figure 2-1) is the southernmost watershed in the study area and traverses Brownsville in a northwest to southeast direction for about 7.75 river miles through the downtown area and outfalls into NMD downstream of the Impala Pump

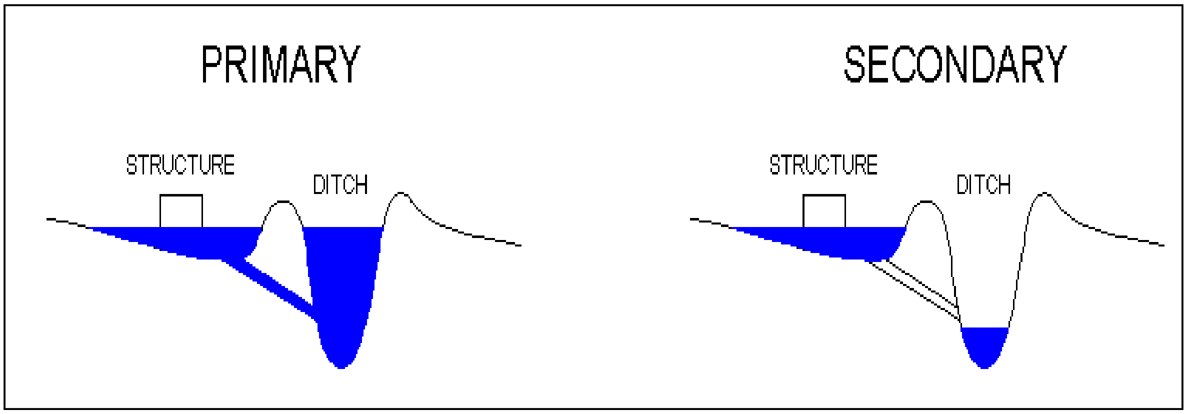
Station. Here, excess water is pumped over the Rio Grande Levee and into the river. The watershed is approximately 5.7 square miles with slopes averaging between 0.4% to 1.6% with some localized values near the resaca banks being as high as 5.6%. The predominant land use in Town Resaca is high intensity residential and commercial/industrial.

2.2.2 Secondary Drainage System

The secondary drainage system in the City of Brownsville consists of storm sewer systems and valley gutters along most of the streets. In addition there are several pump stations throughout the Brownsville area that serve to 1) pump water out of the streets in flood prone areas, 2) feed water to isolated lakes, ponds, and resacas, and 3) transport raw water to the Brownsville Public Utilities Board (PUB) water treatment plants. However, since the flow capacities of these pump stations are negligible when compared with the flows observed in the main system during flood events, they are not included in this study.

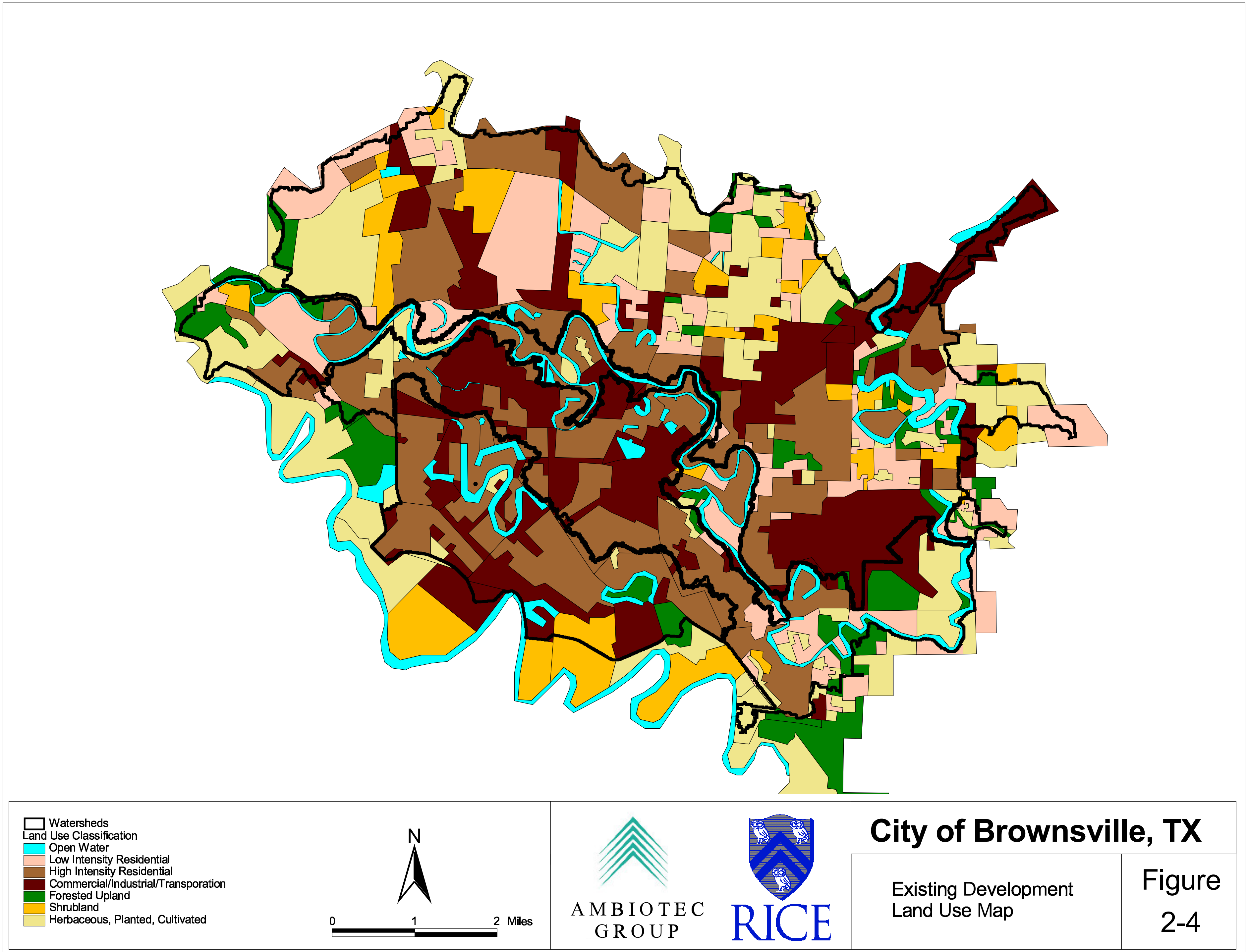
The focus of this study is exclusively on the primary drainage system. As shown in Figure 2-3, there are two general types of flooding experienced in the study area. First, shallow flood plain flooding that occurs when the channel capacity of the primary system is exceeded; and second, ponding/overland flow flooding that occurs when the local rainfall exceeds the capacity of the secondary system. Although secondary system flooding can occur throughout the City, it is short-lived if the primary system has sufficient capacity. However, in the case of Brownsville, the primary system has insufficient capacity to handle even the current levels of development. Fixing the secondary system, without first addressing the primary system, would be counterproductive.

Figure 2-3. Primary and Secondary Flooding



2.3 Land Use Data

Land use data was obtained through visual inspection and field verification of high-resolution aerial photographs provided by Brownsville Public Utilities Board. Polygons were drawn in ArcView around regions of similar land use and assigned a code based on the National Land Cover Data (NLCD) set. A few simplifications were made in the classification scheme based on the inability to distinguish one land cover type from another in certain regions. For example, deciduous forest could not be distinguished from evergreen or mixed forest so a value of 40 was assigned to all forested areas as opposed to a 41, 42 or 43 respectively. Table 2-1 below describes the classification scheme adapted from the NLCD classification study. The existing land use map created for this study is consistent with trends observed by various City officials and may be seen in Figure 2-4. The future land use map (discussed in section 4.3) is shown in Figure 2-5.



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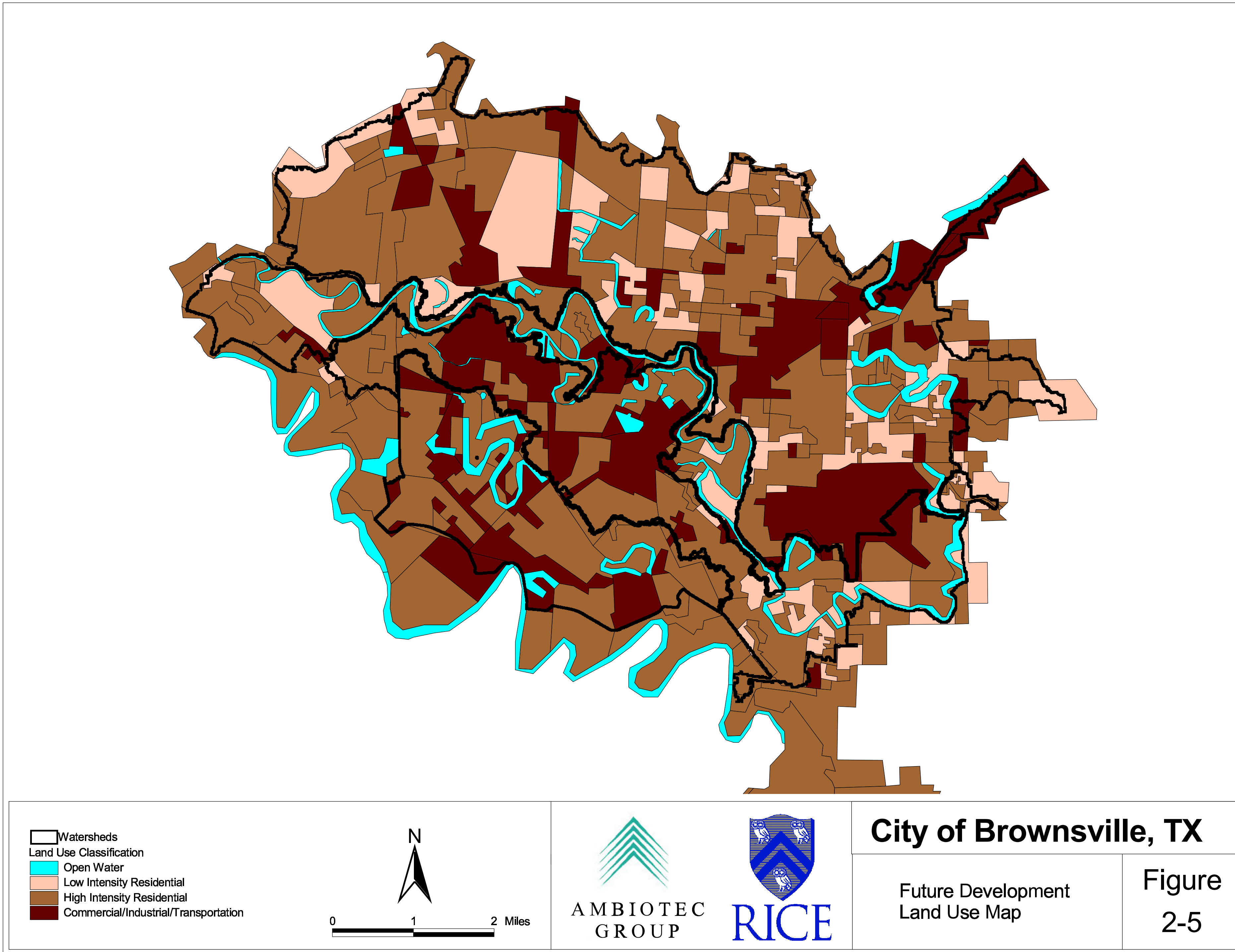
Existing Development
Land Use Map

Figure
2-4



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City of Brownsville, TX

Future Development Land Use Map

Figure 2-5

Table 2-1. Land Cover Classification Key

	Grid Code	Description
Water		
	11	Open Water
	12	Perennial Ice, Snow
Developed		
	21	Low Intensity Residential
	22	High Intensity Residential
	23	Commercial, Industrial, Transportation
Barren		
	30	Bare Rock, Sand/Clay
		Quarries, Strip Mines/Gravel Pits
Forested Upland		
	40	Deciduous Forest
		Evergreen Forest
		Mixed forest
Shrubland		
	50	Shrubland
Non-natural Woody		
	60	Orchards, Vineyards, Other
Herbaceous Upland		
	70	Grasslands, Herbaceous
Herbaceous Planted, Cultivated		
	80	Pasture, Hay
		Row Crops
		Small Grains
		Fallow
		Urban, Recreational Grasses
Wetlands		
	90	Woody Wetlands
		Emergent Herbaceous Wetlands

2.4 Geographical Information Systems (GIS)

Several data sets and files were collected and/or formatted in a GIS for model development and analysis. A more detailed description of certain functions within GIS is described in chapter 3. Included in these data sets are aerial photographs, street data, topographic data, maps of current structures (buildings) in the Brownsville area, and hydraulic structures within the study area, as well as other data that aided in this analysis. In addition to the use of GIS for data collection, it was also used for floodplain delineation and analysis.

2.5 Rainfall Data

Two different types of rainfall data were used for this analysis, design storm rainfall data, and NEXRAD (radar) rainfall data. Each data source will be discussed further in the subsequent sections.

2.5.1 Design Storm Rainfall Data

A design storm is a theoretical precipitation event used as the basis of design for a hydrologic system (Bedient and Huber, 2002). The important factors for defining a design storm are the amount of precipitation and its distribution across the watershed both temporally and spatially. The design storm data used in this study came from the U.S. NWS TP 40 report (Hershfield, 1961). This report presents maps for rainfall durations of 30 min. to 24 hr. and return periods from 1-yr to 100-yr. The return period of the storm is simply the probability of that rainfall intensity occurring within any one-year period. For example, a design frequency of 100 years means that there is a 1%

chance of a storm of that particular intensity occurring in any given year. Discussion with various City officials and other local entities revealed that flooding in Brownsville is not only prevalent for large-scale storms but also smaller more frequent rainfall events. To adequately represent flooding from these varying amounts of rainfall, the runoff response of Brownsville watersheds was analyzed for the 2-yr, 5-yr, 10-yr, 25-yr, 50-yr and 100-yr design storm events. The temporal distribution and 24-hr totals for each of these events is presented in Table 2-2.

Table 2-2. Design storm rainfall totals (inches) for Brownsville, TX.

Rainfall Duration	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1 hour	2	2.8	3.2	3.8	4.2	4.6
2 hours	2.68	3.45	4.08	4.7	5.3	5.9
3 hours	2.9	3.8	4.48	5.25	5.9	6.53
6 hours	3.3	4.6	5.42	6.7	7.3	8.25
12 hours	3.9	5.4	6.48	7.7	8.8	10
24 hours	4.6	6.35	7.48	9	10.3	11.75

2.5.2 NEXRAD Rainfall Data

The second main source of rainfall data came from NEXRAD radar data. While radar technology is not new, hydrologists have only recently begun to apply the technology to hydrologic modeling and real-time flood forecasting (Bedient and Huber, 2002). NEXRAD, or next generation radar, was developed by the National Severe Storms Laboratory (NSSL) in Norman, Oklahoma and is maintained by the National Weather Service (NWS), the Federal Aviation Administration (FAA) and the Air Force.

The NWS deployed its NEXRAD radar systems across the United States beginning in 1992. These radar installations can generate a wide range of data including rainfall estimates, storm tracks, and Doppler velocity. The native resolution of the radar is in polar coordinates of approximately 1° by 1 km. Measurements are taken every 5-6 minutes providing dense spatial and temporal observations over most areas (Crum et al. 1993).

Today such radars exist at approximately 160 locations across the U.S. and abroad at military installations. This network of NEXRAD radars was strategically placed in order to provide full nationwide coverage as well as to provide effective coverage for a variety of meteorological events (Benavides, 2001). The radar tower (KBRO) in the study area is located at the Brownsville airport on the eastern side of the City approximately 2.3 miles away from the outlet of RDLG. While it is generally undesirable to use radar data taken from a tower so close to the study area because of the increased amount of noise, it was the only radar tower close enough to the study area to collect data.

3.0 REVIEW OF COMPUTER MODELS

Modeling of the hydrologic and hydraulic behavior of the primary drainage system was performed by using three models: HEC-HMS, HEC-RAS and Vflo. In addition, ArcView was used to create the floodplains based on the output from the hydrologic/hydraulic models. A summary description of these models is presented below.

3.1 HEC-HMS

The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) is a Windows-based, lumped parameter, hydrologic model that translates rainfall over a watershed into runoff that is then routed through a channel (USACE, 1998). HEC-HMS supersedes HEC-1 and contains many improvements over its predecessor. As these developments progress, it will eventually replace HEC-1. This section contains a brief description of the background, capabilities, and usage of the program; specific information is given on the selection and application of the various methods offered.

The most notable difference between HEC-HMS and HEC-1 is an easy-to-use graphical user interface (GUI), which allows for the manipulation of hydrologic elements such as basin and river reaches and the improved input of basin characteristics. The GUI also allows for the quick viewing of results for any object in the model schematic. A background map containing subwatershed boundaries and streams can be entered from a GIS map file as a visual reference, but it is not used for any calculations.

An advantage of HEC-HMS is the organization of the components, which make up each hydrologic modeling run. In HEC-HMS, a project consists of three separate parts: the Basin Model, the Meteorologic Model, and the Control Specifications. These three parts are accessed by the main screen, called the Project Definition screen, which is the window that initially opens when HEC-HMS is started.

The Basin Model contains the basin and routing parameters of the model as well as connectivity data for the basin. The watershed is represented by a set of subbasins that represent the physical areas within the watershed and produce a discharge hydrograph at the outlet of their respective areas. Hydrographs are computed using one of several transform methods available in the model that convert rainfall excess into surface runoff after taking into account loss rates and base flow. Loss Rates can be simulated by one of several methods. For event modeling, techniques include initial and constant, SCS curve number, gridded SCS curve number, and Green and Ampt methods. Base flow takes into account normal flow through a channel or the effects of groundwater.

Flood routing in HEC-HMS offers a few more options than what was contained in HEC-1. The more popular routing methods include Muskingum, lag, and Modified Puls. The Muskingum method is used for general routing; routing with no attenuation can be modeled with the lag method; and the Modified Puls method is used to model a reach with a user-specified storage-outflow relationship. In addition, Muskingum Cunge Standard, Muskingum Cunge 8-pt, Kinematic Wave, and Straddle Stagger methods are also available in the model for flood wave routing.

Beyond just basic transform and routing methods, HEC-HMS has tools that allow the modeling of a number of other scenarios including reservoirs, sources, sinks, and diversions. A Reservoir in the model stores the inflow from upstream elements and produces an outflow hydrograph based on a storage-outflow relationship. Sources are elements that represent a discharge into the basin as an observed hydrograph or a hydrograph generated by a previous simulation. Sinks are elements that have an inflow and no outflow. Diversions are used for hydrologic models and contain a simple table relating inflow to diverted flow and routed flow.

The Meteorologic Model contains the precipitation data, either historical or hypothetical, for the HEC-HMS model. The new version contains more options than HEC-1 for modeling precipitation and can even account for evapotranspiration. Control Specifications contain all the timing information for the model, including the start time and date, stop time and date, and computational time step of the simulation.

A major difference between HEC-HMS and HEC-1 is the use of the Data Storage System, or HEC-DSS, which is used in HEC-HMS to manage time-series and tabular data. In order to run HEC-HMS and view results, the user may specify different data sets for each component within a project and then run the hydrologic simulation using different combinations of models. HMS includes, or will include, advanced features such as parameter estimation with optimization, soil moisture accounting, GIS and grid cell hydrology, snowmelt simulation, and improved hydraulics.

3.2 Vflo™

Vflo™ is a fully distributed, physics based hydrologic model capable of using geographic information to model the hydrologic response of a watershed. Specifically, the model computes stage and discharge over time throughout the extent of the defined watershed. The model uses a finite element approach expanded to include a network of elements to represent overland and channel flow within a watershed (Vieux, 1988, Vieux et al. 1990, Vieux and Gauer, 1994). An earlier version of this model, r.water.fea, was first developed for the U.S. Army Corps of Engineers, Construction Engineering Research Laboratory (USA-CERL) in 1988. A graphical user interface (GUI) was later incorporated into the model within the ArcView environment. Today, the current version of this model, Vflo™, has been rewritten in Java ™ to take advantage of Servet/Applet technology for multi-user access.

Raster grids, developed in a geographic information system (GIS), can be imported into the model to describe the spatial variability of various rainfall runoff processes. Raster grids can be created at any resolution desired to represent slope, hydraulic roughness, infiltration, precipitation, and flow direction for overland cells. Similarly, channel cells require inputs of slope, bottom width and side slope. Reservoir cells, which require a storage/stage and stage/discharge relationship as an input can be used in the model to represent reservoirs that store water in the system.

With this information incorporated into the model, Vflo™ calculates infiltration excess at each cell in the grid. Based on the finite element connectivity of each cell, a system of equations is then developed for solving the kinematic wave analogy model. The kinematic wave model relies on the principles of conservation of mass and momentum. These equations are then solved using the Galerkin formulation of the finite-element method. Generally speaking, the finite-element method reduces the governing partial differential equations to ordinary equations in time (Vieux and Gauer, 1994). Furthermore the use of a finite-element scheme as opposed to a finite-difference method provides a distinct advantage in that the matrices of the assembled system of equations are banded and symmetric allowing for compact storage and solution methods (Vieux et al., 1990; Vieux and Gauer, 1994). This solution method allows for the calculation of flow rates and depths at every cell in the watershed.

3.3 HEC-RAS

The U.S. Army Corps of Engineers (ACOE) developed the Hydrologic Engineering Centers' River Analysis System (HEC-RAS) in 1994 as an improvement to its predecessor, HEC-2. HEC-2 was first released in 1968 to calculate water surface profiles for steady, gradually varied flow in open channels. HEC-RAS translates peak flow rates computed by the hydrologic model into water surface elevations that can than be used for floodplain delineation. The most recent version of HEC-RAS, version 3.1.1, was released in May 2003 and is capable of modeling steady, one-dimensional, gradually varied flow or unsteady one-dimensional flow. The output of the model includes water surface elevations throughout the watershed based on flows computed from the hydrologic model. These water surface elevations are then used to complete floodplain delineations. Future versions of the model will be developed to include moveable boundary sediment transport analysis.

HEC-RAS, while computationally identical to HEC-2, has been updated with many improvements since the models first release in 1968. The biggest improvement is the addition of a powerful graphical user interface (GUI). The GUI is a system of windows that allows the user to enter, edit, and display data and graphs in an easy to read format. This capability enables the modeler to better visualize the stream and its condition. It even allows for three-dimensional plotting of the stream geometry.

HEC-RAS also includes the ability to model inline weirs and gates and multiple culvert openings, and has a new method for handling piers on bridges. Another useful addition to the model is the ability to import and export GIS data. Cross sections overlying a georeferenced digital elevation model (DEM), landuse data, and shape files representing flowpaths and channel banks can be directly imported into HEC-RAS. Likewise, water surface profiles can be exported back into a GIS and converted to raster grids for floodplain delineation.

HEC-RAS divides the necessary input into two categories: geometric data and flow data. Both can be accessed through the Edit menu in the main program window or on the tool bar. Doing so takes the user into either the Geometric Data Editor or the Steady Flow Editor. Each project has a main project file, which contains a listing of all supporting files associated with that project, including geometry, flow, plan, and output files. A project can hold many different geometry and flow files, and each combination of geometry and flow files that is simulated creates a plan file that saves that combination. Finally, the output of each run is then stored in an output file.

The computations made in HEC-RAS are based on the solution of the one-dimensional energy equation with Manning's equation accounting for the energy loss due to friction. This computational routine is generally referred to as the standard step method. Through these calculations, the model has the ability to calculate water surface profiles while taking into account backwater effects from bridges, culverts, weirs and other obstructions for subcritical, supercritical and mixed flow regimes (Haestad, 2003).

3.4 ArcView GIS

Geographic information systems (GIS) have been used in a variety of environmental applications. Common to all definitions is the concept of linking data with a location in space, or spatial data. The simplest definition of GIS describes its three integral parts: 1) the database, 2) the spatial or map information, and 3) some way to link the two; and includes the necessary resources: the computer, GIS software, and trained users (Clarke, 2001). A more traditional definition describes GIS in terms of "a powerful set of tools for storing and retrieving at will, transforming and displaying spatial data from the real world for a particular set of purposes" (Burrough, 1986).

Regardless of the definition, GIS record observations or measurements that can be thought of as features, activities, or events. A feature is a term from cartography that refers to an item or piece of information placed on a map. Point features have a location (e.g., a rain gage or a benchmark) while line features have several locations strung along the line in sequence (e.g., river or stream). Area features such as watershed or floodplain boundaries consist of lines that form a loop or polygon. Human activities can often be described with geographical patterns and distributions. Population maps, census maps, and urban infrastructure maps (e.g., sewers and water distribution networks) are examples that show these patterns. Event implies something that occurs at a point in time and can be mapped over time (Mitchell, 1999).

ArcView was developed by the Environmental Systems Research Institute (ESRI) in 1992 to be an easy to use, cost-effective GIS software package capable of bringing geographic information management, analysis, and mapping to the personal computer. While ArcView is not as powerful as the earlier released ARC/INFO it is less expensive and easier to use thus preferred by many mid-level users. ESRI released a new software package in April 2001 called ArcGIS 8.1, however, special extensions available for ArcView especially for the field of hydrology and floodplain mapping still make ArcView 3.2 the preferable software package for the purposes of this study. Some of the significant ArcView extensions used in this study will be described below.

3.4.1 Spatial Analyst Extension

ArcView's Spatial Analyst extension provides the user a broad range of powerful spatial modeling and analysis features. More specifically, Spatial Analyst provides tools to create, query, analyze and map cell based raster data. This function is essential when trying to display items that cannot be modeled as vector data such as digital elevation models and gridded rainfall patterns. The Spatial Analyst extension also has the ability to perform integrated raster-vector theme analysis. This allows for the aggregation of properties in a raster theme based on an overlaid vector theme. These tools allow one to produce essential hydrologic data much more rapidly than what was possible before using manual methods. Additional features unique to Spatial Analyst are listed below:

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

3.4.2 3-D Analyst Extension

The 3-D Analyst extension in ArcView provides the user with tools for three dimensional modeling and analysis. This tool is extremely useful for floodplain delineation studies in that the user is able to create, analyze and display surface data with support for triangulated irregular networks (TINs) and simple three-dimensional vector geometry. The TIN provides a three dimensional topographic base map that when combined with HEC-RAS generated water surface profiles, will determine the extent of the floodplain. Other useful features of 3-D Analyst are listed below:

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

3.4.3 Grid Analyst Extension

The Grid Analyst extension is useful for working with gridded or raster data sets in ArcView. Its most advantageous function for the purposes of this study is its ability to convert grids from one projection to another. Vflo™ requires that all imported grids have units of meters. Therefore, because many data sets use English units, this became an essential tool for creating hydrologic models in Vflo™. Other functions of the Grid Analyst extension include:

- Convert image theme to grid theme
- Convert grid theme to image theme

- Extract grid theme using selected graphics
- Extract X, Y, and Z values for point theme from grid theme
- Convert grid theme to XYZ text file
- Draw a X-Section along a polyline
- Subtract a ‘grid minimum value’ from grid theme
- Calculate grids covariance correlation matrices.

3.4.4 Xtools Extension

The Xtools extension was developed for vector spatial analysis, shape conversions and table management. This extension has many utilities that will not be listed here. A few of the more frequently used features for the purpose of this study are highlighted below:

- Clip with polygon(s)
- Intersect themes
- Merge themes
- Union polygon themes
- Convert polygons to polylines
- Convert shapes to graphics
- Convert graphics to shapes.

3.4.5 HEC-GeoHMS Extension

The Geospatial Hydrologic Modeling System (HEC-GeoHMS) is an extension that when used in conjunction with Spatial Analyst within ArcView is very useful for developing hydrologic modeling inputs. The original software, HEC-PrePro was developed in 1997 by the Center for Research in Water Resources of the University of Texas at Austin to use in conjunction with COE's HEC-HMS (Hellweger and Maidment, 1997). HEC-GeoHMS analyzes digital terrain information and transforms it into data sets that can than be imported into a hydrologic model such as HEC-HMS or Vflo™. This terrain pre-preprocessing can be completed in both interactive and batch modes.

3.4.6 HEC-GeoRAS

The HEC-GeoRAS extension was developed in the 90s to process geospatial data for use with HEC-RAS. The extension creates a link between the display and data management capabilities of GIS with a robust hydraulic modeling program. Using an existing digital terrain model (DTM) even a novice ArcView user can create HEC-RAS import files that contain geometric attribute data as well as many other complementary data sets. Currently, the HEC-RAS GIS import file contains user-defined river, reach and station identifiers, cross-sectional topographic elevation lines, cross-sectional bank stations, downstream reach lengths for the left overbank, main channel, and right overbank, and cross-sectional roughness coefficients (USACE, 2003). Hydraulic structures such as bridges, culverts and weirs, are not included in this import file and must be entered directly into the HEC-RAS model. Post hydraulic analysis results generated by HEC-RAS can than be exported back to HEC-GeoRAS and used in conjunction with the Spatial Analyst extension for floodplain mapping.

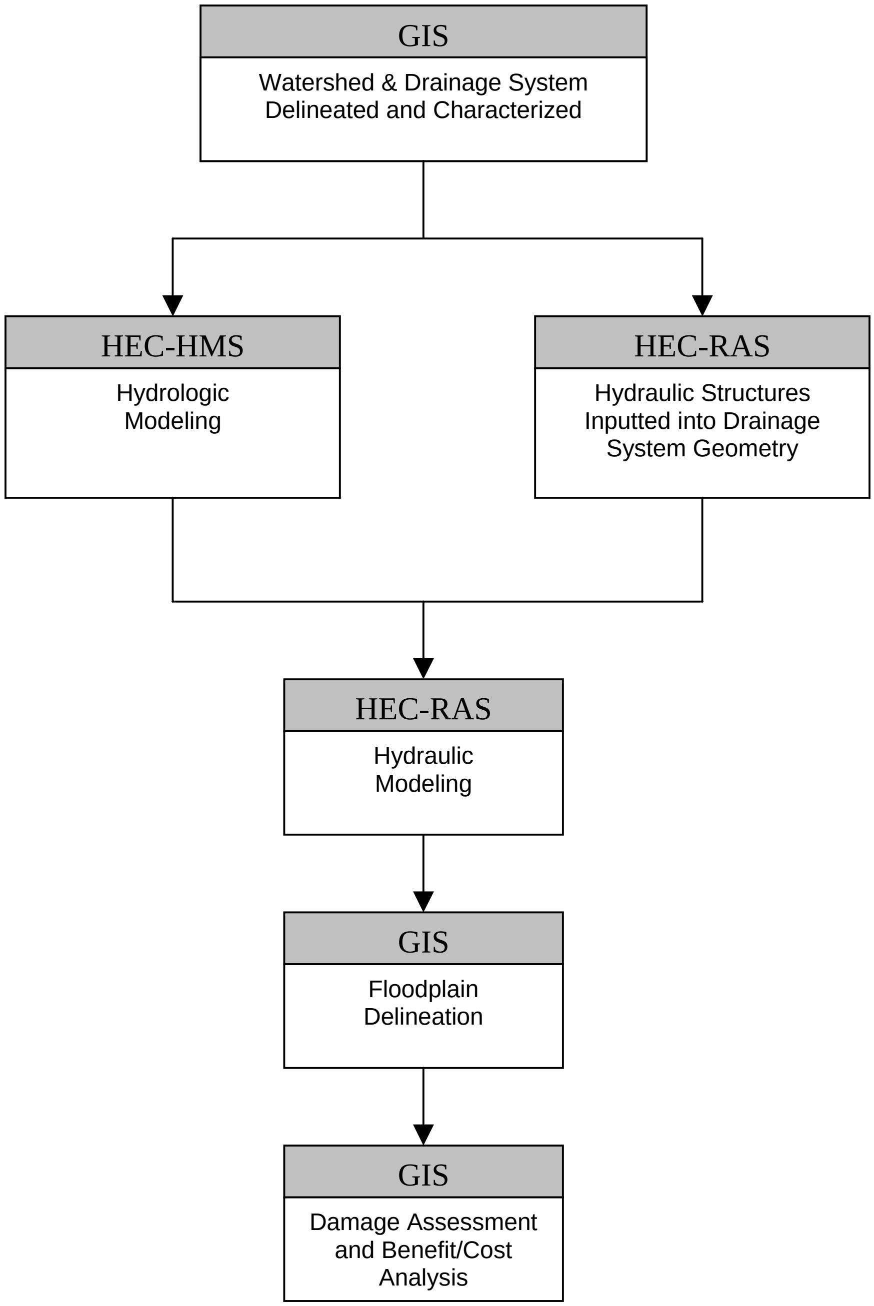
4.0 FLOODING ANALYSIS OF NORTH MAIN DRAIN AND CAMERON COUNTY DRAINAGE DISTRICT NO. 1 DITCH NO. 1

4.1 Analysis of the NMD and CCDD1 Primary Drainage Systems

The North Main Drain and Cameron County Drainage District No. 1 ditch No. 1 drainage systems were analyzed using a five-step process using the ArcView GIS, HEC-HMS, and HEC-RAS models (Figure 4-1) that were described in the preceding chapter. The process included:

- Delineation of watershed and sub-watershed in ArcView GIS in conjunction with the HEC-GeoHMS extension using LIDAR derived topographic data
- Hydrologic modeling of the watersheds in HEC-HMS to simulate peak runoff rates
- Definition of the drainage ditches geometric attribute data using the HEC-GeoRAS extension in ArcView GIS
- Hydraulic modeling of the drainage systems in HEC-RAS to simulate water surface elevations
- Delineation of the simulated floodplains using ArcView with the HEC-GeoRAS extension
- Estimation of flood damage costs and benefits using ArcView GIS based on the number of structures inundated and the level of inundation

Figure 4-1. Flood Analysis Process



A detailed description of the process steps is presented in subsequent sections.

4.1.1 Review of Hydrology

The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used to model the runoff response of the North Main Drain and Cameron County Drainage District No. 1 Ditch No. 1 watersheds. The first step in the creation of the hydrologic model is the delineation of the watershed boundary. The HEC-GeoHMS extension in ArcView GIS was used together with recently collected LIDAR data to create a DEM (digital elevation model) and delineate the watershed. The watersheds derived using this approach are different and more accurate than the delineations used in previous studies, which used the less accurate USGS 5-ft contour maps.

The HEC-GeoHMS extension uses the DEM to analyze the flow direction of water over the terrain. The flow direction algorithm used in this process is referred to as the Eight Direction Pour point Model (Figure 4-2). This algorithm calculates the direction of steepest decent by looking at a 3 x 3 grid of elevation cells. This method assumes that water flows in only one of eight possible directions. Integer values are assigned to each cell in the grid as shown in Figure 4-3. Each integer value represents a direction with 1 representing east, 2 southwest, and so on up to 128 representing flow to the northeast.

To delineate watersheds, stream segments must first be defined as the sections of the stream that connect two successive junctions, a junction and an outlet, or a junction and the drainage divide. Using the stream segment grid and the flow direction grid, sub-watersheds are delineated in a grid representation for each segment. This grid is then converted to vector polygons along with a stream or channel shapefile. The NMD watershed was divided into 18 sub-watersheds ranging in size from 41 to 1147 acres

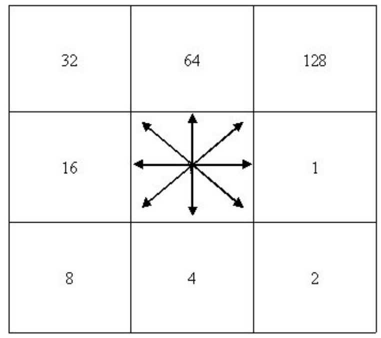


Figure 4-2. Eight Point Pour Directions

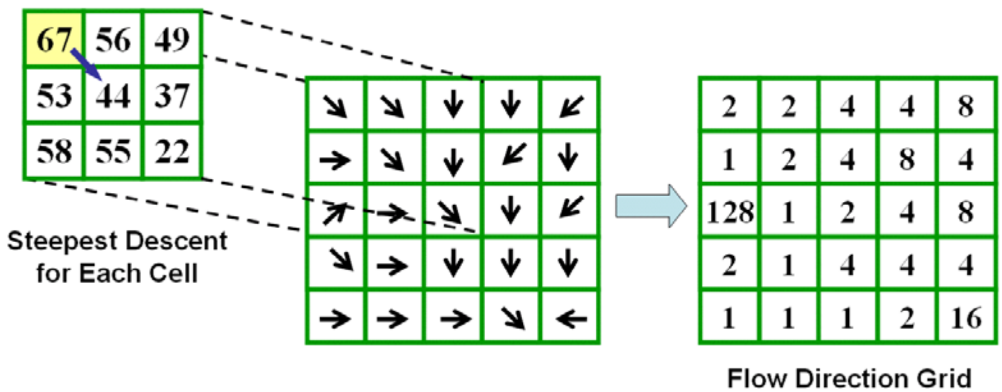
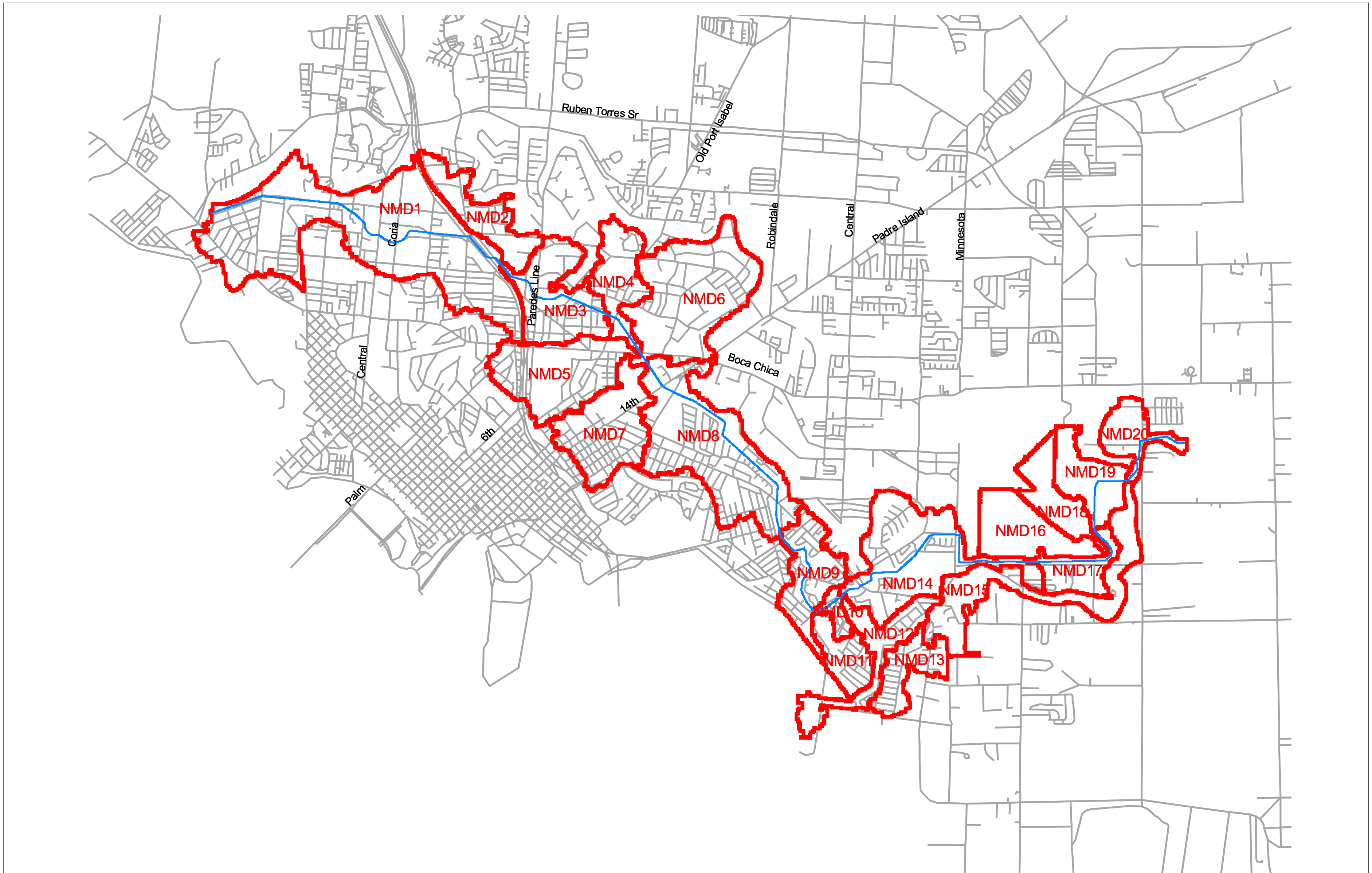


Figure 4-3. Schematic showing the development of a flow direction grid

and the CCDD1 watershed was divided into 27 sub-watersheds ranging from 186 to 1200 acres as shown in Figure 4-4 and 4-5, respectively.

The Clark Unit Hydrograph Transform Method was used to compute the direct runoff from excess precipitation on each of the sub-watersheds (Clark, 1945). The Clark Unit Hydrograph method represents translation and attenuation of rainfall as it moves through the sub-watershed. The required parameters for this method are the time of



NMD
NMD Watershed
Streets

N

0 1 2 Miles

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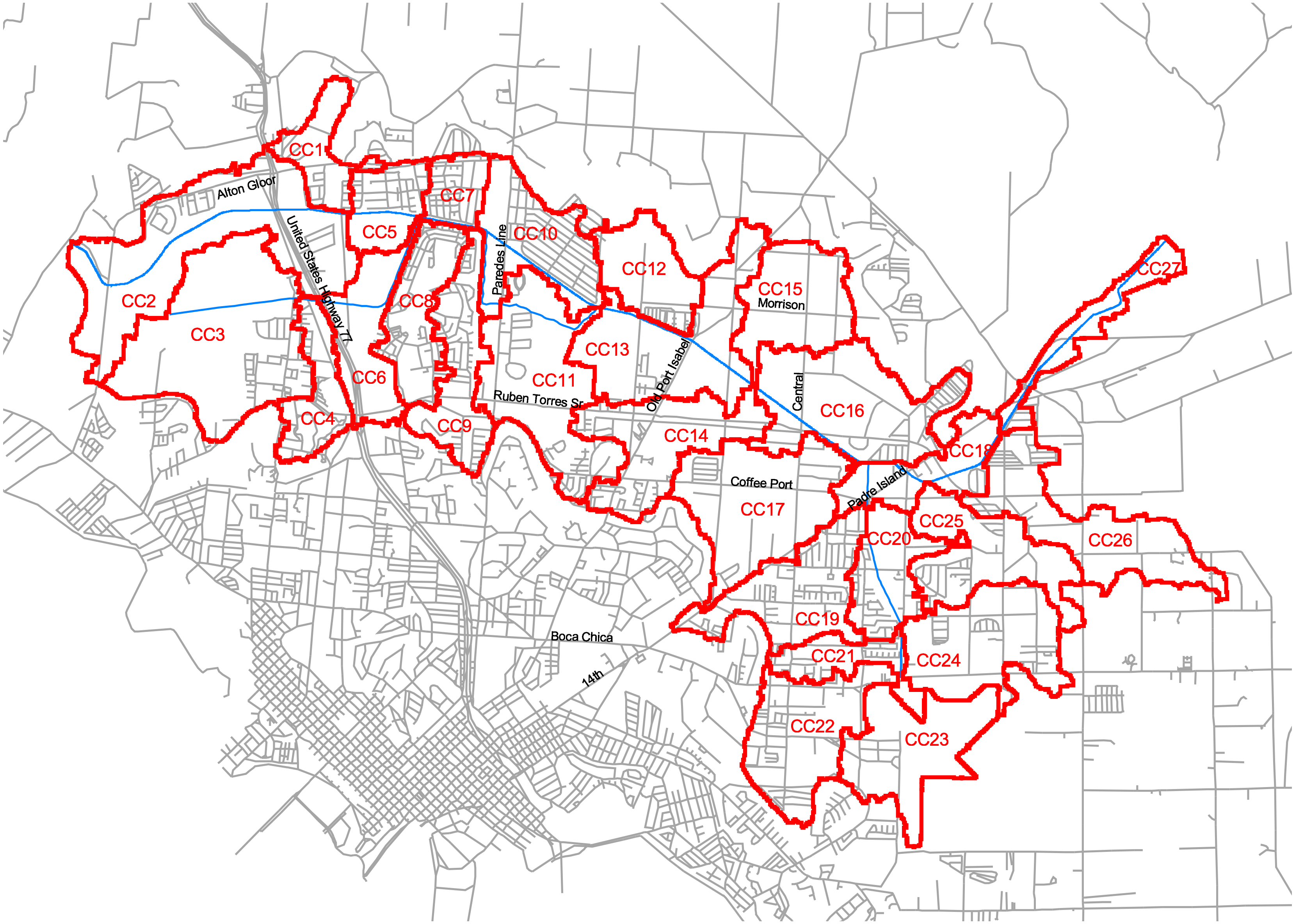
NMD Watershed and
Sub-Watershed Delineation

Figure
4-4



CCDD1
CCDD1 Watershed
Streets

N

0 1 2 Miles

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CCDD1 Watershed and Sub-Watershed Delineation

Figure 4-5

concentration (TC) and the storage coefficient (R), both in hours. TC and R are calculated based on length of channel, channel slope, length along channel to centroid of area, overland slope, percent developed, and percent conveyance. The channel length, channel slope, overland slope, and length along channel to the centroid are calculated by HEC-GeoHMS. Percent developed is based on examining aerial photographs of the watershed, and the percent conveyance is assumed to be 95% for all of Brownsville. The TC & R values for each of the sub-watersheds are listed in Table 4-1 and Table 4-2 for NMD and CCDD1, respectively.

Table 4-1. North Main Drain TC&R values for existing conditions

Subarea	Drainage Area (Acres)	TC (Hours)	R (Hours)
NMD1	1147	0.46	2.43
NMD2	205	0.1	0.64
NMD3	307	0.31	2.23
NMD4	200	0.23	1.13
NMD5	367	0.23	1.17
NMD6	543	0.35	1.77
NMD7	324	0.19	0.95
NMD8	634	1.66	4.5
NMD9	228	0.6	3.07
NMD10	41	0.42	1.73
NMD11	130	1.14	3.34
NMD12	205	0.87	4.24
NMD13	499	0.97	2.62
NMD14	236	0.1	2.03
NMD15	133	2.36	8
NMD16	161	0.16	1.93
NMD17	187	0.29	4.96
NMD18	134	0.23	1.47

Table 4-2. Cameron County Drainage District Ditch No. 1 TC&R values for existing conditions

Subarea	Drainage Area (Acres)	TC (Hours)	R (Hours)
CC1	259	0.6	6.1
CC2	1200	2.2	7
CC3	1161	0.7	7.8
CC4	296	0.3	2.6
CC5	186	0.2	2.7
CC6	417	0.9	5.4
CC7	273	0.3	2.1
CC8	419	0.5	3.2
CC9	458	1.7	6.1
CC10	540	0.7	5.1
CC11	813	1.5	7
CC12	366	0.1	4.2
CC13	785	1.4	11.7
CC14	588	1.1	5.5
CC15	499	1.1	7.8
CC16	804	0.6	7.4
CC17	789	1	4.2
CC18	358	0.4	3.5
CC19	562	0.8	5.1
CC20	371	0.5	3.8
CC21	239	0.3	2.6
CC22	605	0.7	4.4
CC23	586	0.7	5
CC24	663	1	5.8
CC25	564	4.9	7.8
CC26	675	3.3	13.1
CC27	270	1.3	7.8

The flow of water in the open ditches was modeled using the Modified Puls method. The Modified Puls method, also known as storage routing, is based on an approximation of the continuity equation coupled with an empirical representation of the momentum equation. A storage-outflow curve is necessary for each routing reach of the drainage system to calculate the attenuated flood wave.

Design storm rainfall data from 6 different frequency events (Table 4-3) was then entered into HEC-HMS, where the model translated rainfall to runoff in each of the sub-watersheds and along junction points within the drainage ditch.

Table 4-3. Design storm rainfall totals (inches) for Brownsville, TX.

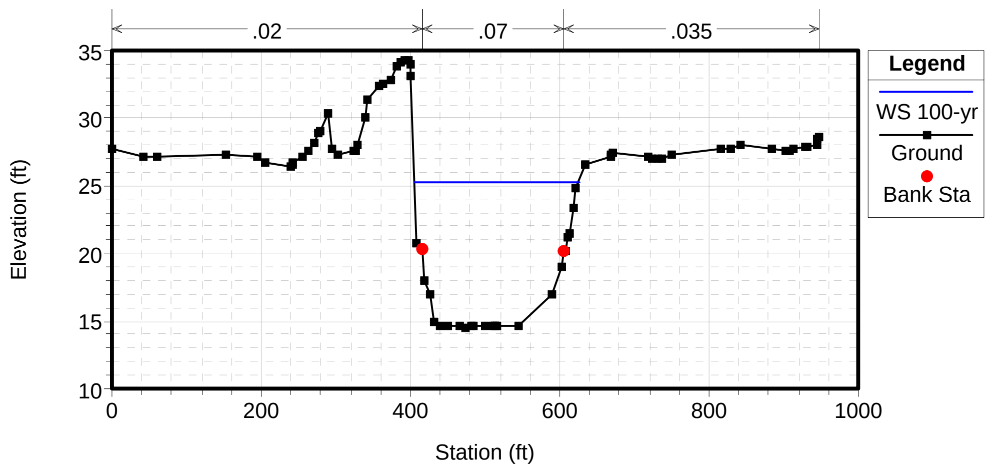
Rainfall Duration	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1 hour	2	2.8	3.2	3.8	4.2	4.6
2 hours	2.68	3.45	4.08	4.7	5.3	5.9
3 hours	2.9	3.8	4.48	5.25	5.9	6.53
6 hours	3.3	4.6	5.42	6.7	7.3	8.25
12 hours	3.9	5.4	6.48	7.7	8.8	10
24 hours	4.6	6.35	7.48	9	10.3	11.75

4.1.2 Review of Hydraulics

A hydraulic analysis of the NMD and CCDD1 was completed through the use of the Hydrologic Engineering Center's River Analysis System (HEC-RAS) as described in section 3.3. HEC-RAS computes water surface elevations for steady, gradually varied flow in open channels. To create this model the HEC-GeoRAS extension in ArcView was used to create a RAS import file. The RAS import file provides the model with geometric data to describe the storage in a channel. Geometric data includes all physical and topographical data entered in a series of cross-sections throughout the watershed.

The first step in the HEC-GeoRAS process is to create a triangulated irregular network, or TIN, from the DEM using the spatial analyst extension in ArcView. The TIN created in this study had a vertical resolution of 1ft and provides topographical data to the RAS model. Using aerial photographs, the stream centerline, flow lines, and bank lines were drawn in ArcView to identify the location of the streams and the direction of flow. The stream geometry data is then completed through the creation of cross-sections that are cut perpendicularly to the channel (Figure 4-6).

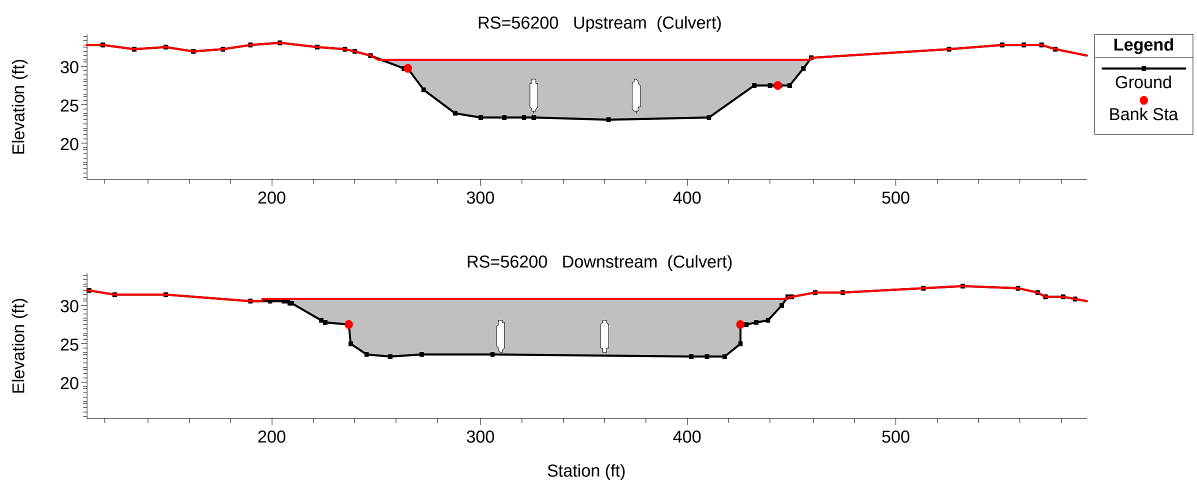
Figure 4-6. A typical Cross-section in HEC-RAS



Each cross-section provides data on elevations, Manning's n, bank locations, and distances to the next downstream cross-section on the left overbank, right overbank and main channel, as well as expansion and contraction coefficients. Typically two cross-sections are placed on either side of a hydraulic structure and where no structures exist, cross-sections are cut every few hundred feet as needed to adequately represent the region. The NMD watershed was characterized with 99 cross-sections and the CCDD1 watershed was characterized with 86 cross-sections. All of this data was then used to create the RAS GIS import file used by HEC-RAS to setup the hydraulic model for developing floodplain information.

After importing the RAS GIS file, information describing the hydraulic structures present in the channel was added to the model. The types of hydraulic structures present in the drainage systems include reinforced concrete pipes (RCP), corrugated metal pipes (CMP), box culverts, weirs, and bridges. Information about each structure including the shape, length, diameter (where applicable), width of deck, distance to the previous cross-section, number of piers (where applicable), flow lines, etc. were entered into the model so that any structures that provide a resistance to flow in the system were represented. Figure 4-7 shows a typical culvert in HEC-RAS. Tables 4-4 and 4-5 show the hydraulic structures in NMD and CCDD1 respectively.

Figure 4-7. A typical culvert in HEC-RAS



The flows modeled from HEC-HMS were then input into HEC-RAS, which computes the water surface elevations throughout the watershed for each design storm event based on peak flow values. These water surface elevations were then imported into

Table 4-4. North Main Drain Hydraulic Structures

Street Name	Station	Type	Number	Size
Kennedy	62537	RCP	1	1'
Midway Dr	61851	RCP	1	1'
Kumquat St	61517	RCP	1	1'
Mesquite St	61169	RCP	1	0.64'
Center Drive	60890	RCP	1	42"
El Pasa Rd	59916	RCP	2	3'
Mopa Rail	59150	CMP	1	4'
Honeydale	57878	RCP	2	3'
Mesquite Grove/Los Sabales	56899	RCP	2	3'
Central	55221	Box	1	6.5' x 4'
Coria	54259	RCP	3	3.5'
West Price	52692	Box	1	8' x 7.1'
Hwy 77	50716	Box	2	8' x 7'
US 83/77	49371	Box	2	8' x 7'
Frontage Road	48014	Box	2	8' x 7'
MacKintosh	47370	RCP	3	5'
Paredes Line	46490	Box	3	6' x 7'
Rockwell	44856	RCP	2	5'
Rentfro	43865	RCP	3	5'
Old Port Isabel	42731	RCP	4	5.5'
Boca Chica	40850	Box	3	10' x 7.77'
Southern Pacific Rail	40070	Bridge	1	
14th Street	39056	Box	3	9' x 9'
International/18th street	38507	Box	3	10.45' x 5.2'
30th Street	34056	Bridge	1	
Southmost	31183	Box	3	10' x 8'
Manzano Street	26984	Bridge	1	
Esperanza	25305	Bridge	1	
La Posada	24450	Box	4	9' x 8'
Ramada	23905	Bridge	1	
Southmost	22646	Bridge	1	
Amatista	19295	Bridge	1	
Minnesota	17295	Bridge	1	
Utah	2252	Bridge	1	
Indiana	2024	Bridge	1	

ArcView where HEC-GeoRAS takes the water depths and overlays them on the TIN to create a grid of water depths over the region. This shows the extent of inundation throughout the watershed as well as providing depths at every grid cell. This is the floodplain delineation map.

Table 4-5. Cameron County Drainage District No. 1 Ditch Hydraulic Structures

Street Name	Station	Type	Number	Size
Alton Gloor Blvd	57176	RCP	1	60"
Railroad	50970	Box	1	8' x 10'
US 77/83	48548	Box	1	6' x 6'
Pablo Kisel Blvd	44820	Box	1	8.7' x 4.5'
Railroad-remnants	39222	Bridge	1	
Paredes Line Rd	38056	Drop Structure	1	4' x 4'
Dana Ave	31011	Bridge	1	
Old Port Isabel Rd	28561	Box	3	10' x 10'
Robindale Rd	26561	Bridge	1	
Central Ave	22536	Bridge	1	
FM 802/Ruben Torres Blvd	21597	Box	3	8' x 10'
Railroad	19258	Bridge	1	
Minnesota and Hwy 48	17000	Box	2	10' x 12'
FM 802/Ruben Torres Blvd	15077	Bridge	1	
Capt. D L Faust	10266	Bridge	1	
FM 511	9430	Bridge	1	
Railroad	8464	Bridge	1	
Hwy 48	6191	Bridge	1	
Hwy 48	3420	Bridge	1	

4.1.3 GIS Floodplain Delineation and Damage Assessment

Floodplains were delineated in ArcView using the HEC-GeoRAS extension. Data files exported from HEC-RAS containing water surface elevation data at every defined cross-section were overlain on a TIN. The difference between the water surface elevations and the land elevation were then used to derive a grid of depths over the watershed area. This grid not only reflects the extent of the floodplain area for a given rainfall event but also illustrates the depths of flooding estimated to occur throughout the watershed.

Estimates of flood damage, in particular expected annual flood damage, are a key element in determining the feasibility of alternative flood damage reduction/prevention options. Since detailed estimation of flood damages for the planning area is outside the scope of this project, approximate (proxy) measures of damage were derived. The proxy estimates of flood damages for a given rainfall event were calculated by determining the number of existing and future structures within the simulated floodplains, and the depth of flooding for each structure. The Brownsville PUB provided a GIS file compatible with ArcView with information about the location of existing structures within the study area. For each structure, the damage for a given rainfall event was adapted from the percent damage functional form used in the Army Corps of Engineer's Hydrologic Engineering Center's Flood Damage Analysis model (HEC-FDA) (USACE, 1998). The amount of damage is given as a percentage of the structure's value as a function of the depth of flooding. The selected function assigned a value of zero damage for zero flooding depth, and a maximum damage of 50% of the structure's value for flood depths greater than 13-ft. For this analysis it was assumed that the average structure value was $60,000.

The total damage for a given rainfall event was obtained by summing the flood damage for all structures within the floodplain. The expected annual damage was then calculated by multiplying the total damage for a given rainfall event by the probability that the given storm would occur in any given year. In order to provide a value comparable to an option's capital investment, the net present value (NPV) of the expected annual damages was then computed using an interest rate of 6% over a 20 year planning horizon.

To assess damages for the full development conditions a grid was created in ArcView placing an average of 2.5 structures on every acre of land excluding any major area that could not support future development (i.e. major streets, water bodies, proposed detention pond sites). This number was based on current land development trends that have been observed in the region. This grid was then overlain on the computed full development floodplains allowing for an analysis of how many structures were likely to be impacted for each design storm and what depth of flooding was likely to occur in each structure.

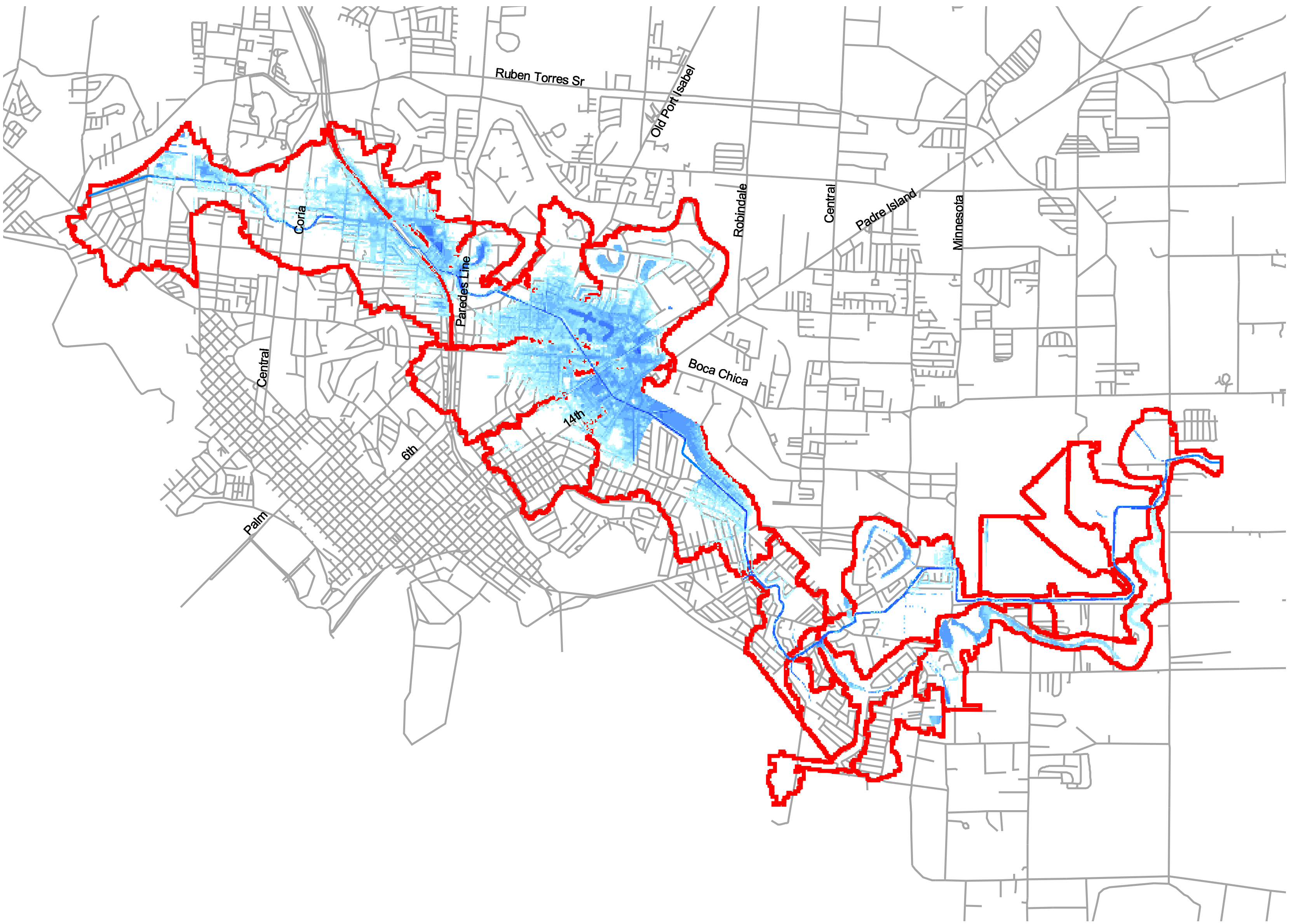
4.2 Existing Development Analysis of North Main Drain and Cameron County Drainage District No. 1 Ditch No.1

The hydrologic and hydraulic models were used to evaluate the existing conditions (current land use conditions) of the NMD and CCDD1 ditches. The models, within the GIS software, were able to locate the areas of out-of-bank flooding and provide the overland water depths to indicate the severity of flooding. The seriousness of the flooding produced by the respective design storms were analyzed according to the extent of the floodplain, the flood depth, and the number of existing structures affected by the floodwaters. The following sections summarize the results of the existing conditions analysis of the NMD and CCDD1 watersheds

4.2.1 North Main Drain

The North Main Drain originates on the western edges of Brownsville, flows for 12 miles through the highly developed sections of downtown Brownsville, past the airport, and then to the Brownsville Ship Channel where it drains a 9.6 square mile total area. North Main Drain, due to its flat topography and undersized channel capacity, experiences severe over-bank flooding problems for all frequency storm events. Presently, there are three major areas of concern. They are located between Coria Street and Paredes Line Road, at the Brownsville airport, and between Rockwell Drive and Esperanza Road. The latter includes the "four corners" area at the intersection of Boca Chica and Hwy 48 where significant flooding problems have been noted in recent years. These regions are of special concern due to the size of the floodplain and the developed nature of the area.

Examination of the various delineated floodplains reveals that there is little difference in terms of lateral extent between the 10-yr, 25-yr, 50-yr and 100-yr floodplains. Figures 4-8 and 4-9 show the similarity of the 10 and 100-year floodplains respectively despite a difference of about 4 inches of rain in a 24-hour period. This is due to the "bowl-like" shape of the basin in this area, as shown in Figure 2-2, caused by the fluvial deposits between the resacas that lie along both sides of the North Main Drain. This causes a deepening rather than a widening of the floodplain. The net effect is that the number of structures within the deeper flooding depth ranges increase dramatically, as seen in Table 4-7. Furthermore, while there is only a modest increase in the overall area of inundation, because this watershed is highly developed with high-density residential neighborhoods, the overall number of flooded structures increases significantly. Overall, approximately 33% of the entire NMD land area is inundated with water for the 100-yr event and over 43% of the area is inundated for the 10-yr event. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix A.



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

NMD
NMD Watershed
Streets

N

0 1 2 Miles

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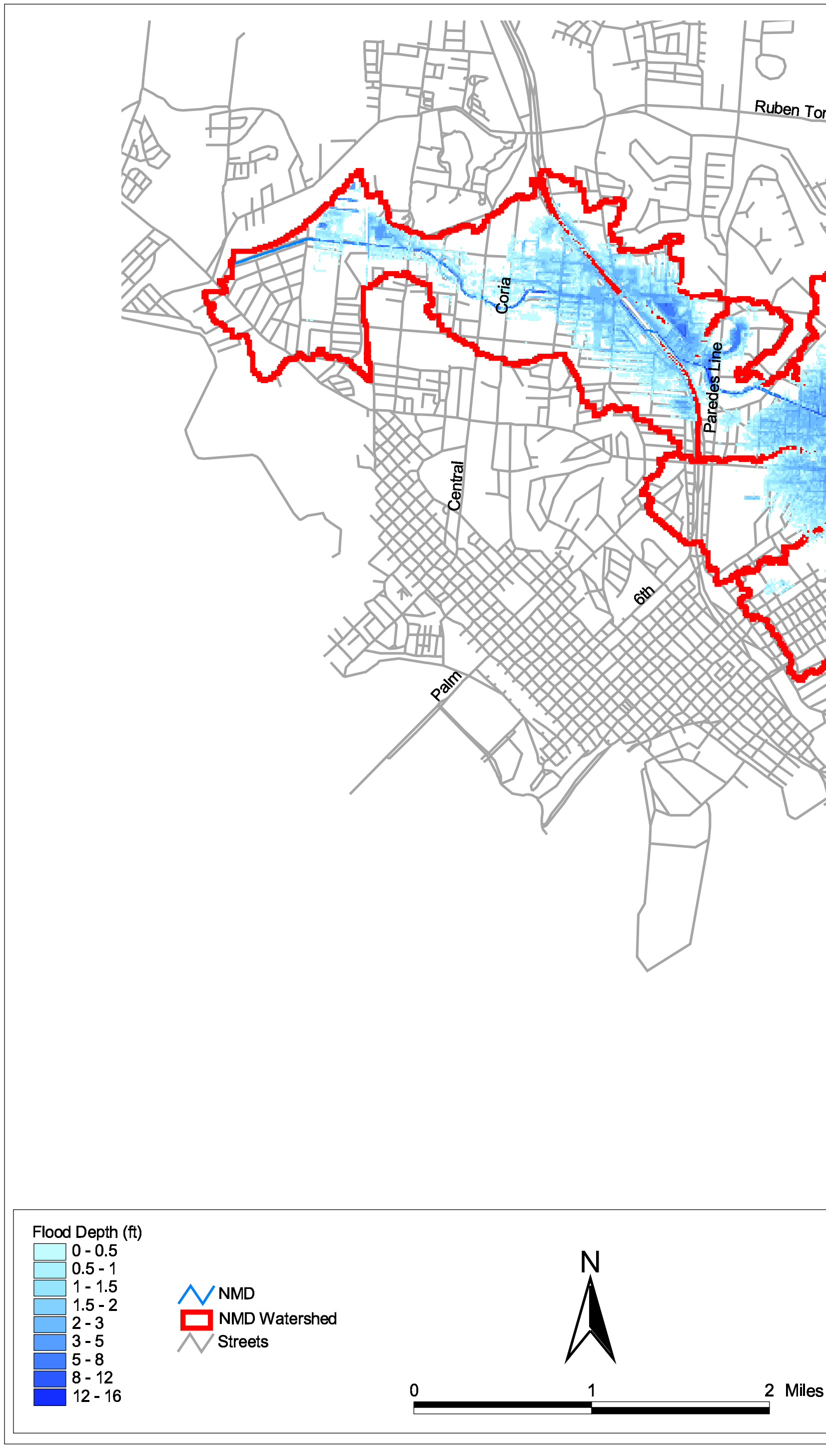
NMD Watershed
Existing Development
10-Year Floodplain Map

Figure 4-8



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NMD Watershed
Existing Development
100-Year Floodplain Map

Figure 4-9

Table 4-6. NMD Existing Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross-section	Profile	Q Total (cfs)	WSE (ft)
Center		60720	10 year	301.57	32.02
			100 year	453	32.31
Mesquite Grove Sub Div	573	56176	10 year	807.79	31.09
			100year	1213.69	31.6
Hwy 77/83	1352	48489	10 year	1795.1	29.81
			100 year	2697.1	30.23
Old Port Isabel	1659	43186	10 year	1507	26.56
			100year	2380.1	27.09
Boca Chica (DS)	2227	40677	10 year	1702.4	26.53
			100year	2320.3	27.06
Btw Boca Chica and 14th	3094	39892	10 year	2661.7	26.53
			100 year	3735.7	27.06
Willow	7375	29506	10 year	2549.4	24.93
			100year	3715.7	26.41
Esperanza	7603	25238	10 year	2485.8	23.31
			100 year	3741	25.11
La Posada (DS)	7849	23862	10 year	2614.5	23.16
			100year	3788.8	24.97
Southmost (DS)	7849	21363	10 year	2613	21.77
			100 year	3788.2	22.75
Minnesota Ave		15438	10 year	2417.6	21.67
			100year	3501	22.97
South Dakota	8348	13070	10 year	2394.4	21.31
			100 year	3398.1	22.57
------	8642	6517	10 year	2042.5	21.07
			100year	2771.3	22.32
------	8829	3619	10 year	2040.2	20.99
			100 year	2768.6	22.25
Utah (US)	8829	2738	10 year	2040.2	20.87
			100 year	2768.6	22.13

Table 4-7. Number of Flooded Structures for Modeled Design Storms

Water Depth (ft)	10-Year	100-Year
0.5 - 1.0	498	855
1 – 1.5	441	671
1.5 - 2.0	256	592
2.0 - 3.0	150	667
3.0 - 5.0	20	210
> 5.0	0	2
Total	1365	2997

4.2.2 Cameron County Drainage District No.1 Ditch

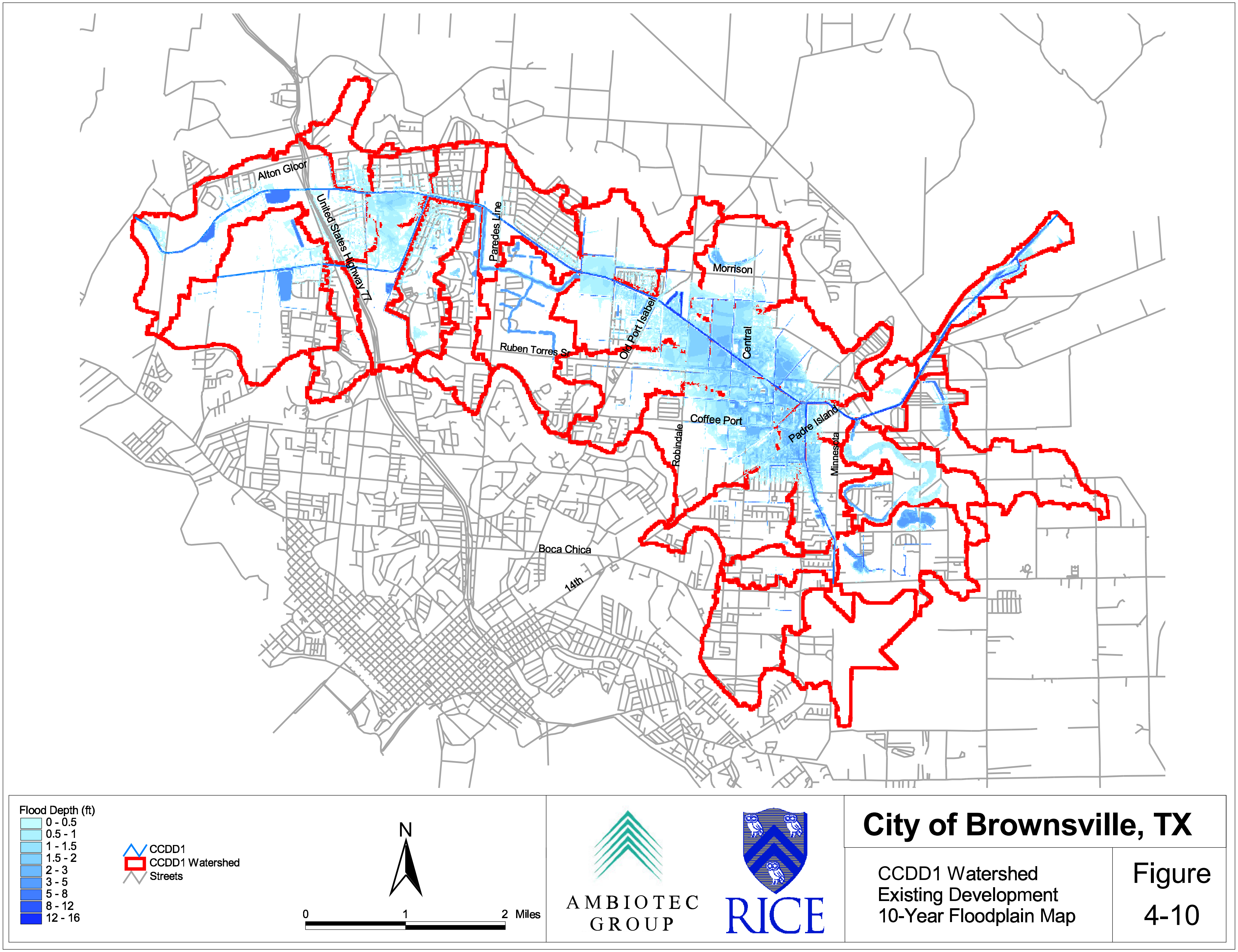
The CCDD1 ditch is located in the northern part of the City of Brownsville and drains a watershed of approximately 23 square miles. The length of the ditch is roughly 11.5 miles, and it extends from Alton Gloor Boulevard in West Brownsville, east across FM 802, then north past the Brownsville Ship Channel where it drains into San Martin Lake, and eventually to the Ship Channel. The Cameron County Drainage District exercises administrative control over this drainage system including channel modifications, maintenance and right-of-way requirements. The watershed is still largely rural but is currently undergoing a rapid increase in development.

Due to the flat topography of the watershed and inadequate capacity of the drainage ditch, the CCDD1 watershed has an extensive flooding problem, mostly affecting undeveloped and agricultural land. Even for small, 2-yr rainfall events (4.6-inches in 24-hr) localized areas of flooding are observed. These areas include the region just west of the Brownsville Country Club (near Pablo Kisel), portions of the region between Robindale and Hwy 48 and small areas within both Brownsville Country Club and Towne North.

As rainfall amounts increase to 6.35-inches in 24-hr (a 5-yr event) the flooding in the aforementioned regions expand and deepen in the surrounding low-lying areas. However because of the “bowl-like” nature of the surrounding topography, larger storm events cause little expansion to the already significant floodplain but rather cause a deepening of floodwaters in already impacted areas. Evidence of this may be observed in Figures 4-10 and 4-11, which illustrates the 10-yr and 100-yr floodplains respectively, and Table 4-8, which illustrates flows and water surface elevations for the 10-yr and 100-yr events (Appendix A, P-2). Overall, for the 100-yr storm event over 24% of the entire land area is inundated (nearly 17% for the 10-yr event).

The problem in this area is much like the problem in the North Main Drain watershed. The fluvial deposits from Resaca del Rancho Viejo to the north and Resaca de la Guerra to the south of CCDD1 create a shallow valley causing a build up of water. The problem is amplified because CCDD1 cuts through the Resaca del Ranch Viejo fluvial deposits; as a result the only outlet for the floodwaters in the “valley” is the CCDD1 ditch which has insufficient capacity for the volume of water in this area.

The number of flooded structures that result from these two events and the depth to which they are flooded may be viewed in Table 4-9. While these numbers are not as high as what was seen in the NMD watershed, it should be noted that the CCDD1 watershed is still largely undeveloped, and these numbers, as discussed in the next section, will increase greatly as the area continues to develop.


Alton Gloor
United States Highway 77
Paredes Line
Morrison
Old Port Isabel
Central
Ruben Torres Sr
Coffee Port
Padre Island
Robindale
Minnesota
Boca Chica
14th
Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16
CCDD1
CCDD1 Watershed
Streets
N
0
1
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Miles
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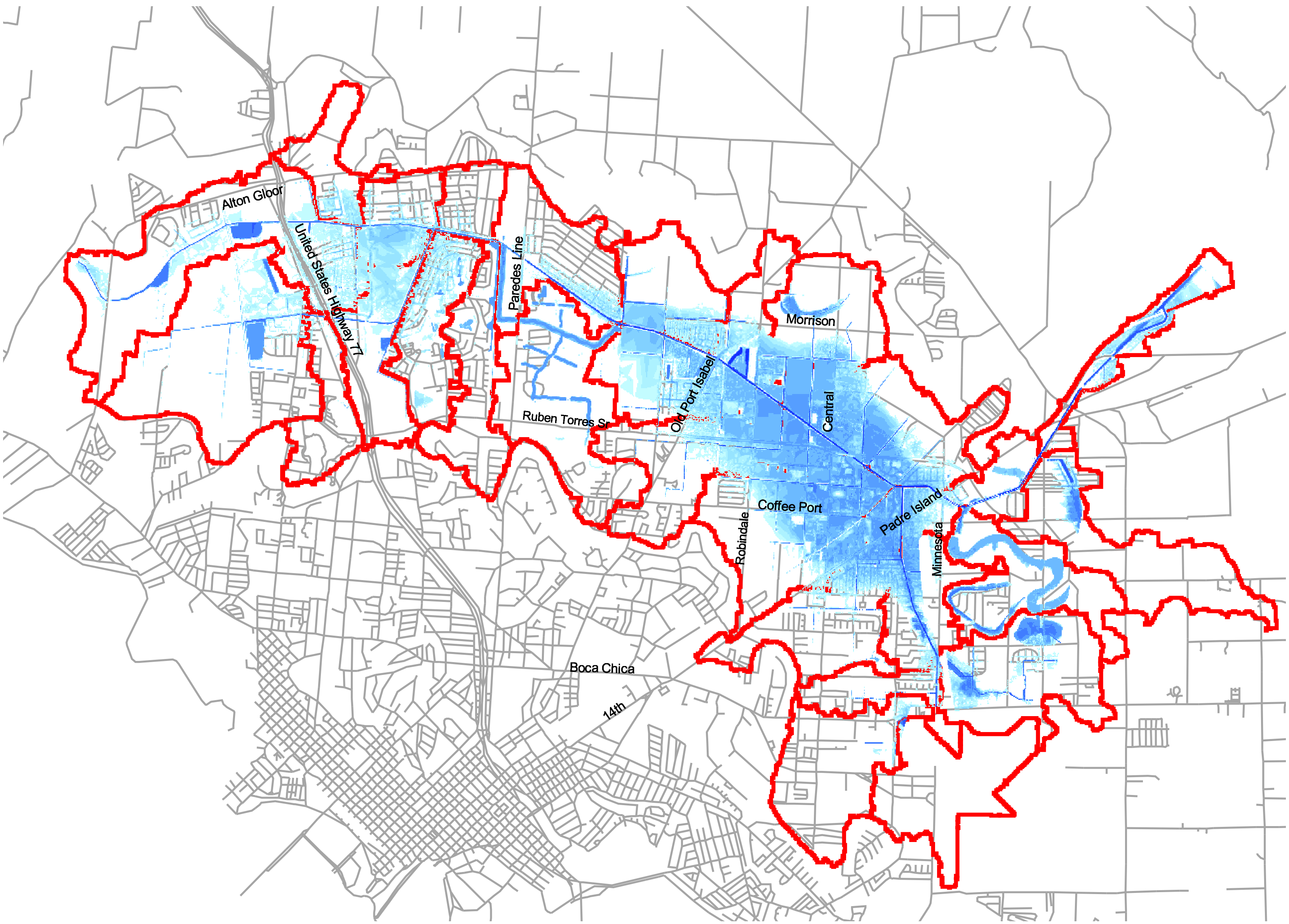
City of Brownsville, TX
CCDD1 Watershed
Existing Development
10-Year Floodplain Map
Figure
4-10



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

- CCDD1
- CCDD1 Watershed
- Streets

N

0 1 2 Miles

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City of Brownsville, TX

CCDD1 Watershed
Existing Development
100-Year Floodplain Map

Figure 4-11

Table 4-8. CCDD1 Existing Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross-section	Profile	Q Total (cfs)	WSE (ft)
Beginning	0	59386.47	10 year 100 year	1 1	27.24 27.35
Alton Gloor	159	57023.95	10 year 100 year	224 354	22.94 23.45
HW 77	913	48213.56	10 year 100 year	672 1062	21.39 21.86
Pablo Kisel	1200	44494.17	10 year 100 year	896 1415	20.94 21.42
Paredes Line	4667	37764.99	10 year 100 year	500 800	17.77 18.72
Dana Ave	6386	30766.94	10 year 100 year	2810 4353	17.32 18.41
Old Port Isabel	6779	28290.97	10 year 100 year	2810 4353	16.98 18.20
Robindale	7171	26203.98	10 year 100 year	2810 4353	16.90 18.17
Central Ave	8258	22326.12	10 year 100 year	3052 4551	16.80 18.13
FM 802	9527	21389.29	10 year 100 year	3052 4551	16.78 18.11
Padre Is. & Minnesota	13192	16499.11	10 year 100 year	4464 6045	16.35 17.99
FM 802	13756	14785.25	10 year 100 year	4612 6100	16.00 17.77
Capt. Donald L Faust	14591	10256.28	10 year 100 year	4780 6343	11.36 12.89
FM 511	14655	9272.439	10 year 100 year	4780 6343	10.94 12.51
Port Isabel Hwy	14719	6391.508	10 year 100 year	4780 6343	9.12 10.22

Table 4-9. CCDD1 Flooded Structures for Modeled Design Storms

Water Depth (ft)	10-Year	100-Year
0.5 – 1.0	207	511
1 - 1.5	103	344
1.5 – 2.0	84	290
2.0 – 3.0	30	277
3.0 – 5.0	8	83
> 5.0	0	2
Total	432	1507

4.3 Full Development Analysis of North Main Drain and CCDD1

Urbanization generally increases the volume and peak rates of runoff as the land is covered by more impervious surfaces (e.g., paved roads and parking lots) and natural drainage is replace by storm sewer systems (Bedient and Huber, 2002).

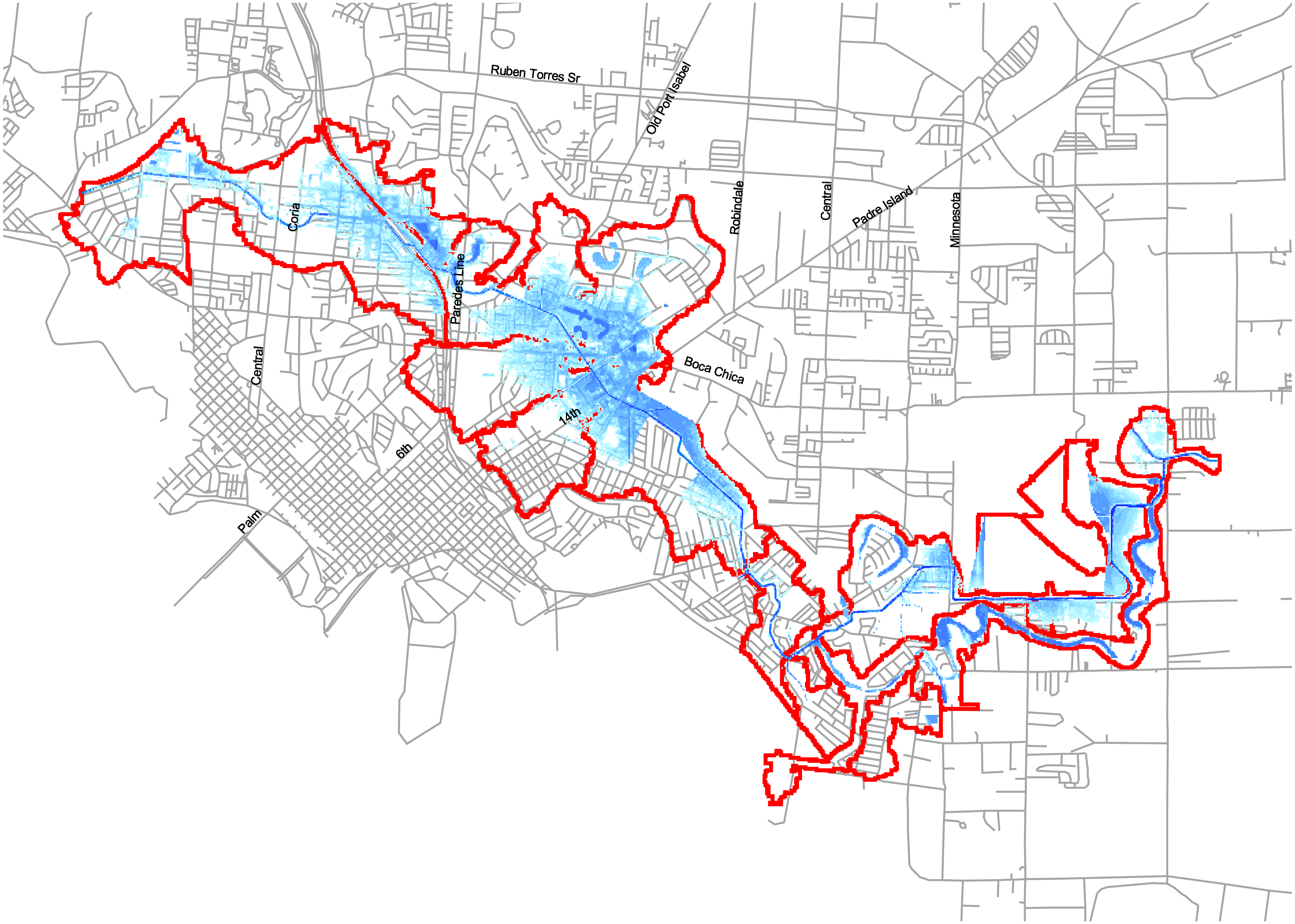
To determine the impact of future development on the NMD and CCDD1, the existing hydrologic and hydraulic models were revised to reflect a projected future land use scheme. The assumption made when creating a projected future land use map was that any undeveloped parcel of land would be developed into a high-density residential area; and that any area already developed would remain in its current land use category. This is consistent with development trends observed throughout the area as was discussed with various City officials and local parties. The TC &R values in the hydrologic model for NMD and CCDD1 were recalculated based on this new land use map. Tables 4-10 and 4-11 show the new TC &R values for the ditches and Figures 4-12, 4-13, 4-14 and 4-15 show the corresponding floodplains for full development. The effects of using full development watershed parameters in the models will be discussed individually in the following sections. Water surface elevations may be viewed in Appendix A (P-3-P-4).

Table 4-10. NMD Full Development Hydrologic Parameters

Subarea	Drainage Area (Acres)	Tc (Hours)	R (Hours)
NMD1	1147	0.4	2.29
NMD2	205	0.05	0.64
NMD3	307	0.27	2.1
NMD4	200	0.15	0.87
NMD5	367	0.18	1.03
NMD6	543	0.25	1.49
NMD7	324	0.18	0.92
NMD8	634	1.43	4.31
NMD9	228	0.52	2.9
NMD10	41	0.33	1.59
NMD11	130	0.63	2.17
NMD12	205	0.44	2.3
NMD13	499	0.65	2.17
NMD14	236	0.03	0.64
NMD15	133	0.95	2.36
NMD16	161	0.07	0.6
NMD17	187	0.12	1.55
NMD18	134	0.13	0.93

Table 4-11. CCDD1 Full Development Hydrologic Parameters

Subarea	Drainage Area (Acres)	Tc (Hours)	R (Hours)
CC1	259	0.3	2.8
CC2	1200	1.3	5.0
CC3	1161	0.3	3.7
CC4	296	0.3	2.5
CC5	186	0.1	1.9
CC6	417	0.6	4.7
CC7	273	0.3	2.1
CC8	419	0.5	3.2
CC9	458	1.6	6.0
CC10	540	0.6	4.6
CC11	813	0.8	4.8
CC12	366	0.1	2.5
CC13	785	0.7	6.6
CC14	588	0.7	4.0
CC15	499	0.5	3.2
CC16	804	0.3	4.1
CC17	789	0.7	3.6
CC18	358	0.3	3.1
CC19	562	0.6	4.3
CC20	371	0.3	3.2
CC21	239	0.3	2.4
CC22	605	0.7	4.4
CC23	586	0.7	5.0
CC24	663	0.6	4.2
CC25	564	2.6	4.8
CC26	675	1.9	9.0
CC27	270	0.5	4.0



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

NMD
NMD Watershed
Streets

N

0 1 2 Miles

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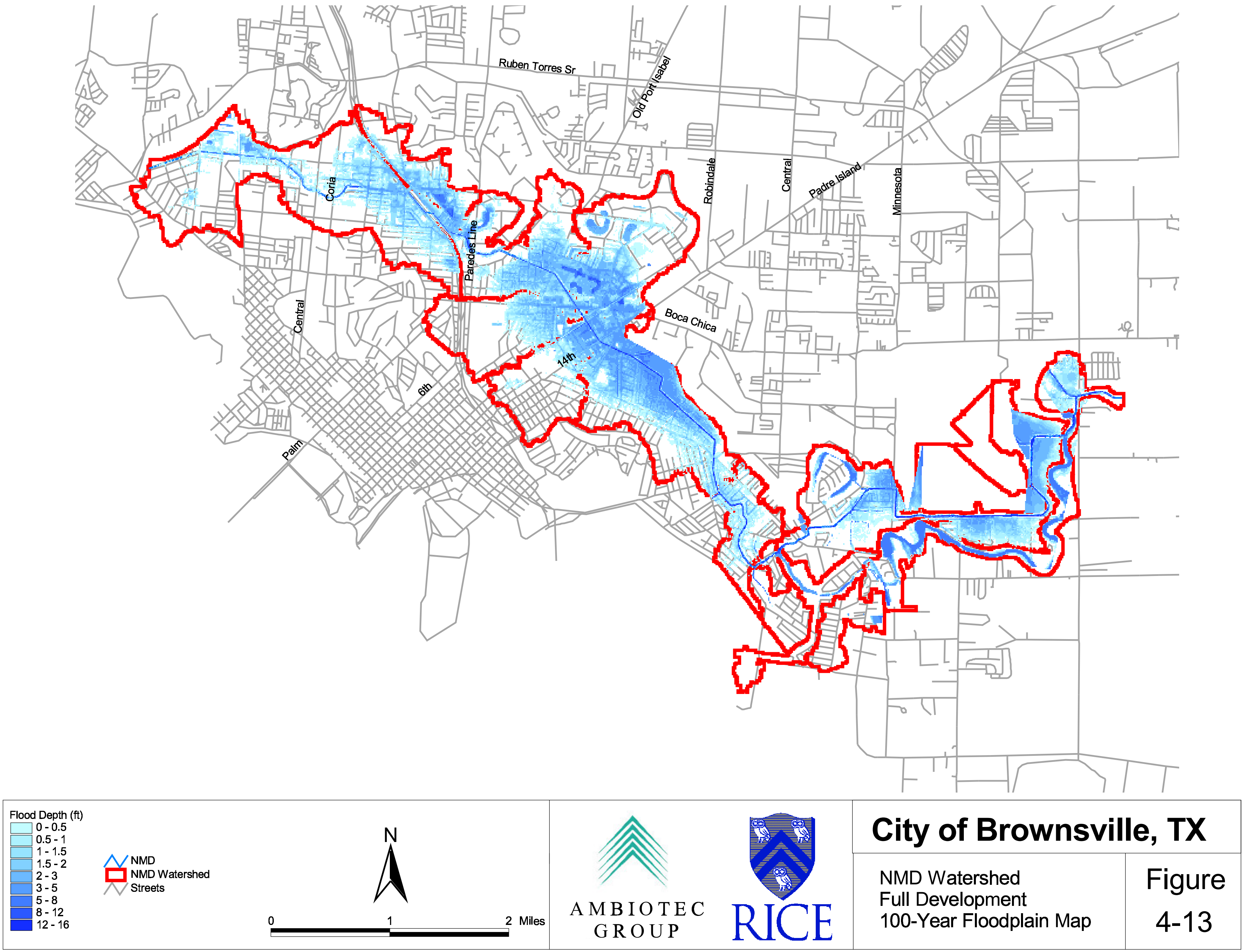
NMD Watershed
Full Development
10-Year Floodplain Map

Figure 4-12



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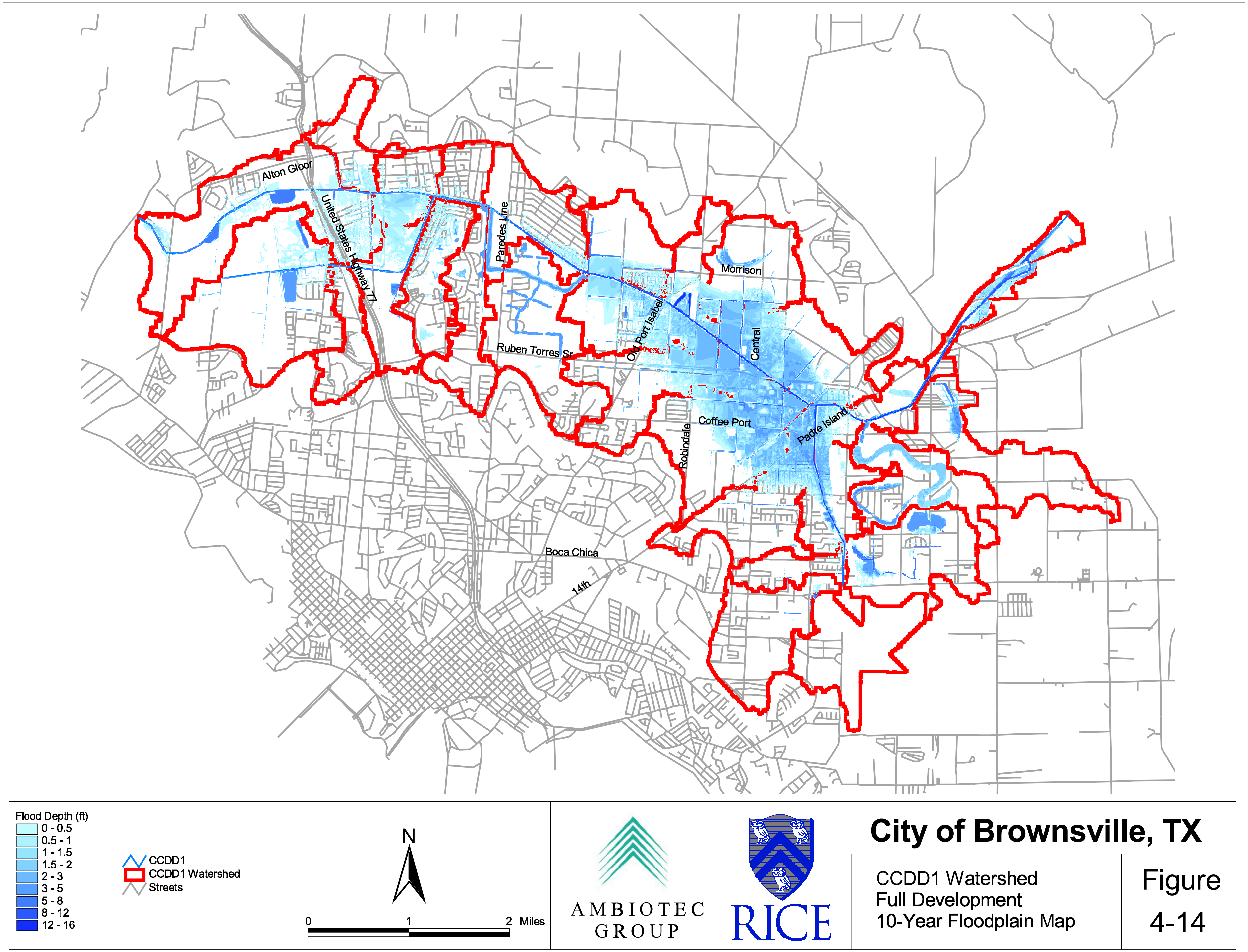
NMD Watershed
Full Development
100-Year Floodplain Map

Figure 4-13



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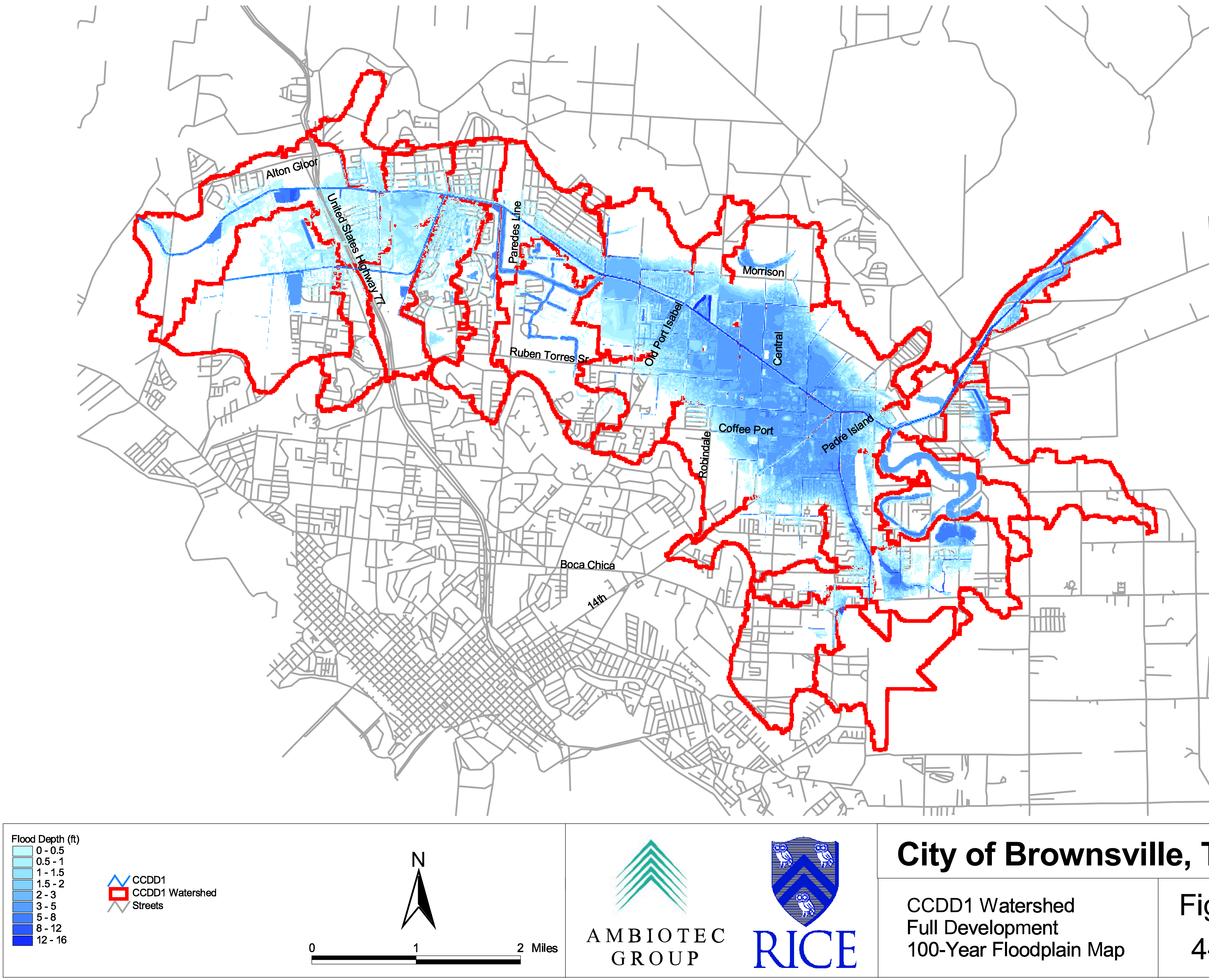
CCDD1 Watershed
Full Development
10-Year Floodplain Map

Figure 4-14



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City of Brownsville, TX

CCDD1 Watershed
Full Development
100-Year Floodplain Map

Figure 4-15

4.3.1 North Main Drain

Since the majority of the North Main Drain watershed is already highly urbanized, there are only small differences between existing and full development TC&R values, flows, or water elevations until moving closer to the outlet (Tables 4-10 and 4-12). An examination of the total out of bank flooding volume that results from the 10-yr and 100-yr events (Table 4-13) reveals a 46% and 25% increase in out of bank flooding volumes between the existing and future land use models respectively. This occurs despite the fact that there is no significant increase in the total floodplain area between the existing and full development models. This is a reflection of the increase of water depths in the existing floodplain area caused by the projected future development and mostly occurs in the downstream portion of the watershed.

Table 4-14 shows the number of structures impacted by the full development 10-yr and 100-yr floodplains relative to that of existing development conditions. These figures were computed by assuming that the entire land area, with the exception of areas excluded from development, would be covered with 2.5 structures per acre during full development. This table shows a greater significant increase in the floodplain between the existing and developed conditions for the 10-yr event than for the 100-yr event. In the case of the 100-yr event, the increase in the floodplain volume and area are 25% and 2%, respectively. Again, in the case of the 100-yr event, the difference between the existing and fully developed case would be reflected in the depth of flooding.

Table 4-12. NMD Future Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross-section	Profile	Q Total (cfs)	WSE (ft)
Center		60720	10 year 100 year	315.96 473.54	31.94 32.17
Mesquite Grove Sub Div	573	56176	10 year 100year	843.35 1263.92	31.06 31.49
Hwy 77/83	1352	48489	10 year 100 year	1873.3 2807.4	29.84 30.27
Old Port Isabel	1659	43186	10 year 100year	1541.2 2465	26.44 27.07
Boca Chica (DS)	2227	40677	10 year 100year	1793.3 2444.1	26.43 27.05
Btw Boca Chica and 14th	3094	39892	10 year 100 year	2838.7 3964.2	26.43 27.05
Willow	7375	29506	10 year 100year	2569.7 3750.7	24.64 26.33
Esperanza	7603	25238	10 year 100 year	2490.8 3750	22.71 23.99
La Posada (DS)	7850	23862	10 year 100year	2696 3777.3	22.5 23.63
Southmost (DS)	7850	21363	10 year 100 year	2693.7 3776.9	21.81 22.86
Minnesota Ave		15438	10 year 100year	2442.5 3505.6	21.65 22.71
South Dakota	8348	13070	10 year 100 year	2422.1 3407.2	21.46 22.53
------	8643	6517	10 year 100year	2079.2 2790.3	21.34 22.4
------	8829	3619	10 year 100 year	2077.2 2787.7	21.29 22.33
Utah (US)	8829	2738	10 year 100 year	2077.2 2787.7	21.18 22.22

Table 4-13. Total out of bank flooding volume for the 10-yr and 100-yr storm events in NMD

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	1807	2046	3443	2674
Full Development	2630	2076	4296	2699
% Increase	46%	2%	25%	9%

Table 4-14. Number of structures within the 10-yr and 100-yr floodplains for NMD assuming a 2.5 house/acre density for the future development scenario

	Existing Development		Future Development			
Water Depth (ft)	10-Year	100-Year	10-Year	% Increase	100-Year	% Increase
0.5 - 1.0	498	855	585	17%	740	-13%
1 - 1.5	441	671	478	8%	634	-6%
1.5 - 2.0	256	592	399	56%	552	-7%
2.0 - 3.0	150	667	429	186%	751	13%
3.0 - 5.0	20	210	186	831%	396	89%
> 5.0	0	2	82	-	118	5800%
Total	1365	2997	2160	58%	3191	6%

4.3.2 Cameron County Drainage District No. 1 Ditch No.1

The full development land use analysis for CCDD1 revealed increased flows and water surface elevations over the existing conditions throughout the entire of watershed (Table 4-15). Flows near the middle of the watershed reached values up to 40 percent higher than for the existing conditions model for all frequency storm events. This increase in flow does not noticeably affect the extent of the floodplain, but it causes a dramatic increase in the depth of the floodplain of more than 2 feet in the downstream section of the ditch for the 10 and 100-year storm events. This increase in depth causes a significant increase in the total out of bank flooding predicted despite the relatively small change in the overall extent of the floodplain as seen in Table 4-16.

The increased elevation resulting from the full development scenario will cause extensive damage to future buildings if no action is taken to control or alleviate the effects of rainfall runoff. The Full Developed model for CCDD1 increases the number of flooded buildings by a factor of approximately 11 for the 10-Year storm and a factor of 5 for the 100-Year storm. Table 4-17 shows the number of flooded structures under the full Development scenario. The large increase for the 10-Year storm is of major concern because storms of this magnitude are frequent in Brownsville and will result in greater flooding depths for existing buildings.

Table 4-15. CCDD1 Future Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross-section	Profile	Q Total (cfs)	WSE (ft)
Beginning	0	59386.47	10 year	1	27.32
			100 year	1	27.47
Alton Gloor	159	57023.95	10 year	315	23.27
			100 year	485	23.76
HW 77	913	48213.56	10 year	945	21.77
			100 year	1456	22.12
Pablo Kisel	1200	44494.17	10 year	1260	20.92
			100 year	1941	21.29
Paredes Line	4667	37764.99	10 year	500	18.24
			100 year	800	19.28
Dana Ave	6386	30766.94	10 year	3547	17.81
			100 year	5353	19.02
Old Port Isabel	6779	28290.97	10 year	3547	17.54
			100 year	5353	18.9
Robindale	7171	26203.98	10 year	3547	17.51
			100 year	5353	18.87
Central Ave	8258	22326.12	10 year	3636	17.45
			100 year	5134	18.85
FM 802	9527	21389.29	10 year	3636	17.44
			100 year	5134	18.84
Padre Is. & Minnesota	13192	16499.11	10 year	4791	17.23
			100 year	6437	18.78
FM 802	13756	14785.25	10 year	4863	16.99
			100 year	6106	18.66
Capt. Donald L Faust	14591	10256.28	10 year	4991	13.51
			100 year	6298	14.77
FM 511	14655	9272.439	10 year	4991	12.38
			100 year	6298	13.47
Port Isabel Hwy	14719	6391.508	10 year	4991	10.62
			100 year	6298	11.19

Table 4-16. Total out of bank flooding volume for the 10-yr and 100-yr storm events in CCDD1

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	3227	2462	6459	3650
Full Development	4634	3053	8793	4338
% Increase	44%	24%	36%	19%

Table 4-17. Number of Flood Buildings For Modeled Design Storms in CCDD1 for Full Development (2.5 Buildings/Acre)

	Existing Development		Future Development			
Water Depth (ft)	10-Year	100-Year	10-Year	% Increase	100-Year	% Increase
0.5 - 1.0	207	511	1179	470%	1492	192%
1 - 1.5	103	344	901	775%	1179	243%
1.5 - 2.0	84	290	861	925%	913	215%
2.0 - 3.0	30	277	995	3200%	1553	461%
3.0 - 5.0	8	83	368	4500%	1774	2037%
> 5.0	0	2	296	-	451	22400%
total	432	1507	4600	965%	7362	389%

5.0 FLOODING ANALYSIS OF TOWN RESACA AND RESACA DE LA GUERRA

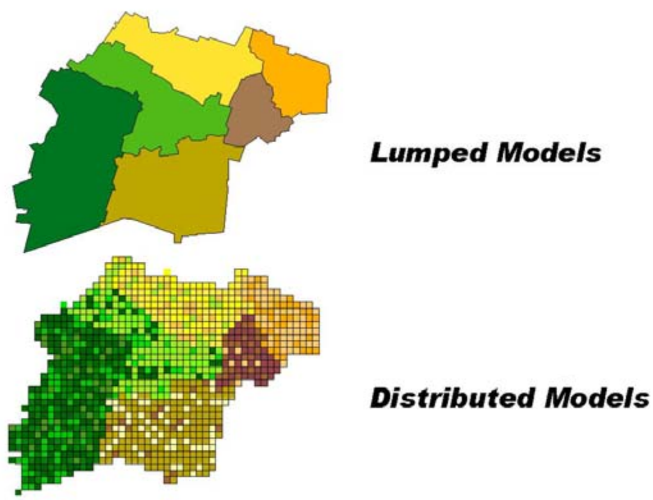
5.1 Analysis of the Existing Primary Drainage Systems

The Resaca de la Guerra and Town Resaca watersheds were analyzed using the same 5-step approach that was used for the two drainage ditches with one exception. Instead of using HEC-HMS for the hydrologic model, a distributed, physics-based hydrologic model, Vflo™, was used to simulate the runoff-response of the two watersheds in conjunction with HEC-RAS and ArcView GIS. This model was used instead of the HEC-HMS model due to the complex nature of the hydraulically controlled resaca pools. This model is described in greater detail in section 3.2.

5.1.1 Review of Hydrology

Vflo™, a distributed, physics-based hydrologic model was used to simulate the runoff response of Resaca de la Guerra (RDLG) and Town Resaca (TR). The HEC-GeoHMS extension in ArcView GIS was again used for setup of the hydrologic model. LIDAR-derived topographic grids were used to re-delineate watershed boundaries around the two resacas and thus differ from those used in previous studies. The Eight Direction Pour point Model algorithm described in section 4.1.1 was then used to delineate the flow direction grid from the digital elevation model. From this, watershed and sub-watershed boundaries were delineated using the same method described for CCDD1 and NMD. Sub-watershed boundaries are not required for the Vflo model, but were delineated for tracking purposes only. As opposed to the HEC-HMS model that averages watershed parameters over each sub-area, Vflo™ preserves the spatial variability of these parameters to a scalable grid size. Examination of Figure 5-1 reveals six different sub-areas for the sample watershed displayed next to the lumped model approach. Each of these sub-areas would be described with one set of averaged parameters that control the runoff-response of this area. The distributed parameter approach however (bottom of Figure 5-1) breaks these sub-areas down further into a network of grid cells, all of which contain their own set of parameters to describe the runoff response of the watershed within that area. All grids used for this model were scaled to a 60-m resolution.

Figure 5-1. Spatial variability in lumped verses distributed models



The key input for development of the hydrologic model in Vflo™ is the flow direction grid. This grid shows the path that a drop of water would take to the resaca where it is eventually routed down to the outlet, if it fell over any portion of the watershed. This grid is directly imported into the Vflo™ model along with other grids that describe the runoff-response of the area including slope and roughness coefficients derived from the land use maps. To distinguish from overland and channel cells a grid identifying channel cells is created in ArcView and imported into Vflo™. These channel cells are described by a channel width, side slope, channel roughness, and bottom slope.

In addition to overland cells and channel cells a third type of cell, a reservoir cell, was used in the model to simulate the affects of flow-restricting culverts in the system. Reservoir cells are characterized by a storage-stage and stage-discharge relationship. These cells allowed for the simulation of flow from one resaca pool to the next by limiting the amount of discharge that could pass through that cell at a given stage based on the size and type of culvert present at a given location.

The same design storm rainfall data that was used for the CCDD1 and NMD models was entered into the Vflo™ model and run for the six different design storms.

5.1.2 Review of Hydraulics

The hydraulic analysis of RDLG and TR was completed using the HEC-RAS model as done for CCDD1 and NMD (section 4.1.2). The same procedure for the HEC-RAS model setup that was used for the drainage ditches was used for the two resaca systems. However, unlike the drainage ditches the resacas have a permanent pool maintained by weir structures. Due to this factor the LIDAR-derived topographic data does not reflect the bottom depth of the resaca but an elevation within the upper depths of the water. To correct this measurement, cross-section data collected by the U.S. Corps of Engineers in 2003 was used to adjust the elevations within the banks of the resacas in the HEC-RAS model.

Table 5-1. Resaca de la Guerra Hydraulic Structures

Street Name	Station	Type	Number	Size
Alton Gloor	84060	RCP	1	36"
Laredo	80650	RCP	1	30"
Laredo	75700	RCP	1	24"
FM 802	73700	RCP	2	48"
Laredo	73160	RCP	2	30", 24"
Mercedes Rd	69800	Weir	1	29.2' high
Railroad	67600	Bridge	1	
Golf Course	66300	RCP	2	18"
Golf Course	65000	Bridge	1	
Golf Course	63883	RCP	1	24"
Golf Course	63668	RCP	2	24"
Golf Course	63000	RCP	1	18"
Golf Course	62840	RCP	1	24"
Golf Cart Bridge	60700	Bridge	1	
Golf Course/Old Hwy 77	60225	RCP?	1	42"
Old Highway 77	60200	Bridge	1	
Central Blvd	59700	RCP	1	48"
US 83/77	57800	Box	1	5' x 5'
N/A	57000	Weir	1	29.4' high
Old Alice	56200	RCP	2	52"
Hidden Valley Drive	53150	RCP	2	30"
Remnants of railroad bridge	50680	posts		
N/A	50100	Weir	1	27.4' high
Paredes Line	49300	RCP	1	52"
Palo Verde Drive	45600	RCP	1	36"
N/A	37000	Weir	1	26.1' high
Old Port Isabel	36150	Box	2	8' x 8'
N/A	28000	Bridge	1	
Price Rd	27300	Box	1	10' x 8'
Padre Island Highway	24600	Weir	1	23.2' high
Padre Island Hwy	24600	RCP	1	70"
Boca Chica	15300	Box	2	10' x 8'
Boca Chica	15285	weir	1	22.5'
Billy Mitchell Blvd	13300	RCP	3	42"
Acacia Lake Drive	11000	Bridge	1	
Morningside Rd	4400	RCP	3	1, 15" and 2, 30"
Morningside Rd	200	RCP	3	30"
N/A	60	Drop structure	1	6' x 6'
Shidler	3800	RCP	1	24"
Price Rd	1700	RCP	1	24"
Eagle	300	RCP	1	24"
Owens Rd	800	Bridge	1	

The RDLG watershed was represented with 201 cross-sections over an approximately 17-mile long resaca, and the TR watershed was modeled with 100 cross sections over its 7.5-mile length. These cross sections provide the model with a description of the geometry of the channel in terms of station, elevation, and roughness coefficients. These cross-sections along with the stream centerlines, flow lines, and bank lines were merged with the TIN to create a RAS GIS import file. The HEC-RAS software reads this file to create the hydraulic model. Flow-restricting hydraulic structures are then added to the model. RDLG contains 35 hydraulic structures and TR contains 28 as shown in Tables 5-1 and 5-2 respectively.

Peak flow values computed by the Vflo™ model were translated into water surface elevations by the HEC-RAS model by taking into account backwater effects caused by the various hydraulic structures. Water surface elevations are computed for each cross- section and this data is then imported into ArcView GIS where the HEC-GeoRAS extension overlays this information on a topographic data set and computes a grid of depth values over the area. This floodplain grid shows the extent of the inundated area as well as the depth of water in each respective area.

Table 5-2. Town Resaca Hydraulic Structures

Street Name	Station	Type	Number	Size
Los Ebanos Blvd.	39450	RCP	1	18"
N/A	39089	Weir	1	29.9' high
Central Blvd.	37600	RCP	1	18"
Coria St.	36800	RCP	1	15"
Boca Chica Blvd.	35150	RCP	1	24"
Belthair St.	33700	RCP	1	18"
N/A	31788	Weir	1	27.1' high
Calle Retama	30300	Box	1	10' x 8'
Pedestrian Bridge	28100	Box	1	12' x 6.5'
Ringgold St.	26400	Box	1	10' x 10'
Calle Retama	25500	Box	1	10' x 8'
N/A	25100	Box	1	9' x 4'
Palm Blvd.	24700	Box	1	10' x 6'
Palm Blvd	24450	weir	1	25.64'
Old Alice	22600	Box	2	9' x 4'
Railroad Crossing	22100	Box	3	8' x 10'
Ringgold St.	20300	Weir	1	22.53' high
6th St.	17200	Box	2	9' x 9'
7th	17111	box	2	10' x 8'
US 83/77	16756	box	2	10' x 8'
Railroad Crossing	15800	Box	2	10' x 8'
12th St.	15400	Box	2	10' x 9'
13th St.	14000	Box	2	10' x 9'
14th St.	13600	box	2	10' x 9'
International Blvd	12285	box	2	10' x 9'
Father Ballard	11000	Bridge	1	
Weir	10800	weir	1	19.95'
US 83/77	10400	Bridge/Weir	1,1	20' high
WWTP Facility Crossing	4700	2 RCP, 1 CMP	3	2, 36" 1, 80"
East Ave.	4500	Bridge	1	
US 83/77	3800	Bridge	1	
Impala Drive	2600	Bridge	1	
Calle Milpa Verde Dr.	1700	Bridge	1	
Tulipan	400	Bridge	1	

5.2 Existing Development Analysis of Resaca de la Guerra and Town Resaca

The hydrologic (Vflo™) and hydraulic (HEC-RAS) models that were created were then used to analyze the existing conditions (current land use conditions) of Resaca de la Guerra and Town Resaca. The data generated by the models was examined and analyzed within a GIS framework to locate areas of out-of-bank flooding, and to identify which structures or buildings within the watershed area was likely to be damaged as a result of flooding. The following sections summarize the results of the existing conditions analysis of Resaca de la Guerra and Town Resaca.

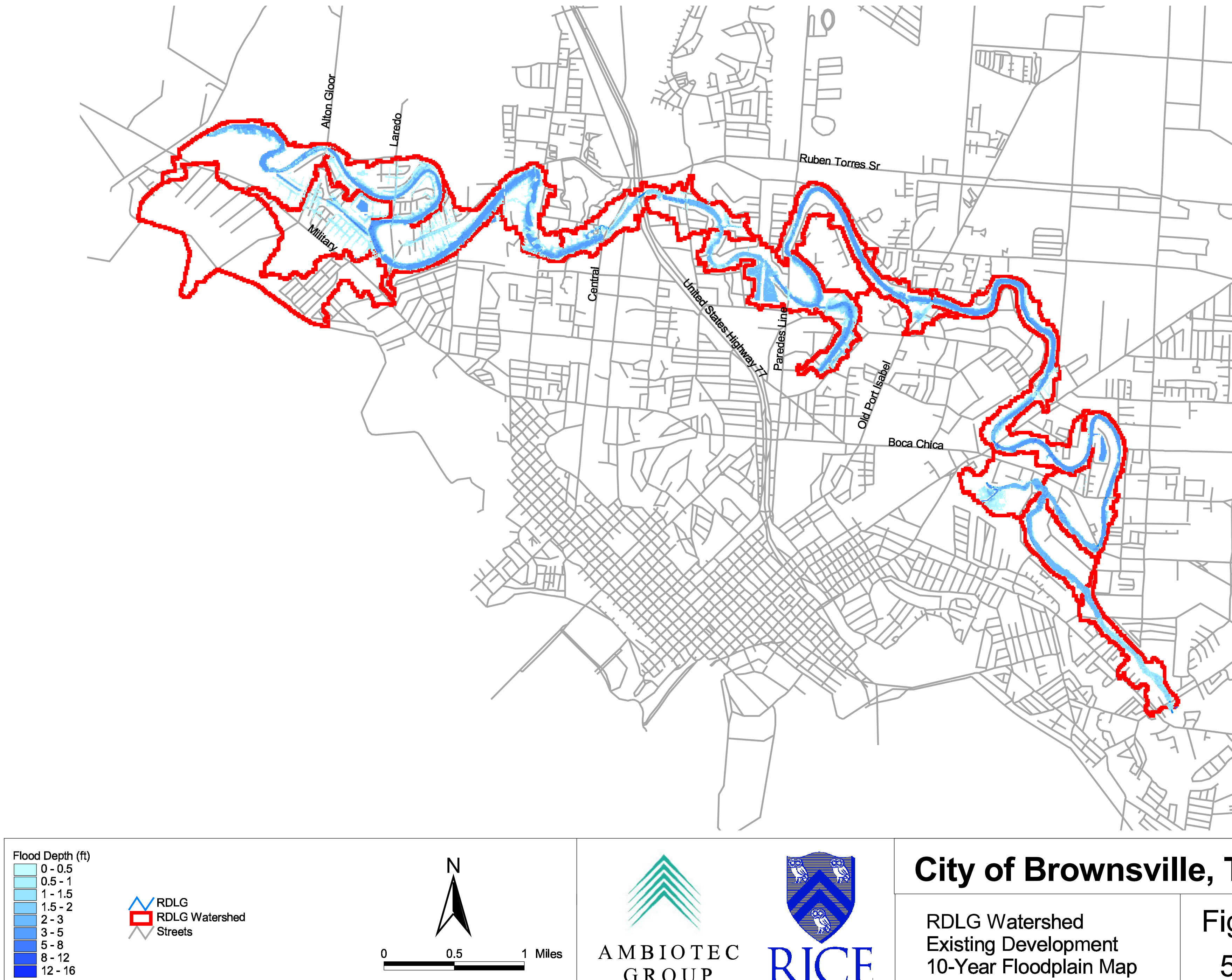
5.2.1 Resaca de la Guerra

Resaca de la Guerra (RDLG) flows generally in a southeasterly direction starting on the western edge of the city, flowing for approximately 17 river miles through highly residential areas in central Brownsville before draining into North Main Drain. The North Main Drain then routes this flow to the Brownsville Ship Channel. Overall, the floodplains delineated from the six different design storms did not vary significantly in terms of the extent of flooding. The exception to this is in the upper portion of the watershed where the floodplain area noticeably expanded for each frequency event. This is likely due to culverts in this area that are capable of conveying smaller frequency events but back up for larger rainfall events. Throughout the rest of the watershed only small differences in the floodplain area were observed in localized areas. Overall, approximately 28% of the RDLG land area is inundated with water for the 100-yr event and nearly 23% is inundated for the 10-yr event. The majority of the impact from increasing intensity events was observed in the depth of flooding that occurred within already inundated regions (Figures 5-2 and 5-3). Due to the similarity of the various floodplains and predicted water surface elevations (WSE), only the 10-yr and 100-yr events will be discussed and displayed. The computed WSE's for the other frequency events may be viewed in Appendix B (P-5). Table 5-3 shows the predicted flows and water surface elevations for the 10-yr and 100-yr design storm events for the existing conditions in RDLG.

While flooding around the Resacas is not as significant as was the case with the two drainage ditches, some out of bank flooding was predicted. The greatest amount of out of bank flooding occurs in the uppermost portion of the watershed between Alton Gloor and Laredo Rd. In this region a significant amount of street flooding is observed for all frequency storm events. Located immediately east of the crossing of Laredo Road (Quail Hollow area) over RDLG is a low-lying area that is inundated for all frequency events, with the inundation becoming more severe as rainfall intensities increase. While the number of structures that fall within the 100-yr floodplain is comparably less than as what was predicted for the NMD or CCDD1, there are still approximately 35 structures that would be damaged in a 100-yr event under existing conditions.

Another area of concern within the Resaca de la Guerra watershed is in the vicinity of the crossing of Price Road over an offshoot of RDLG. This area is impacted for all frequency events and results in the inundation (of varying depths) of approximately a dozen structures for all frequency events. The last area of significant flooding within RDLG is located just southeast of the Four Corners area (intersection of Boca Chica and Padre Island Highway) (Figures 5-2 and 5-3). This is an extremely low area that is partially inundated for all frequency events with water depth approaching 3-ft for the 100-yr event. There is also an area near Owen’s Rd in which the property there has experienced repeated flooding over the years. This area will be examined for the possibility of property buy-outs due to the repeated flood events that have occurred there. While other areas exist throughout the watershed where floodwaters exceed the banks, these areas do not pose an imminent threat to surrounding structures.

Table 5-4 illustrates the total number of structures impacted by the 10-yr and 100-yr events in RDLG.



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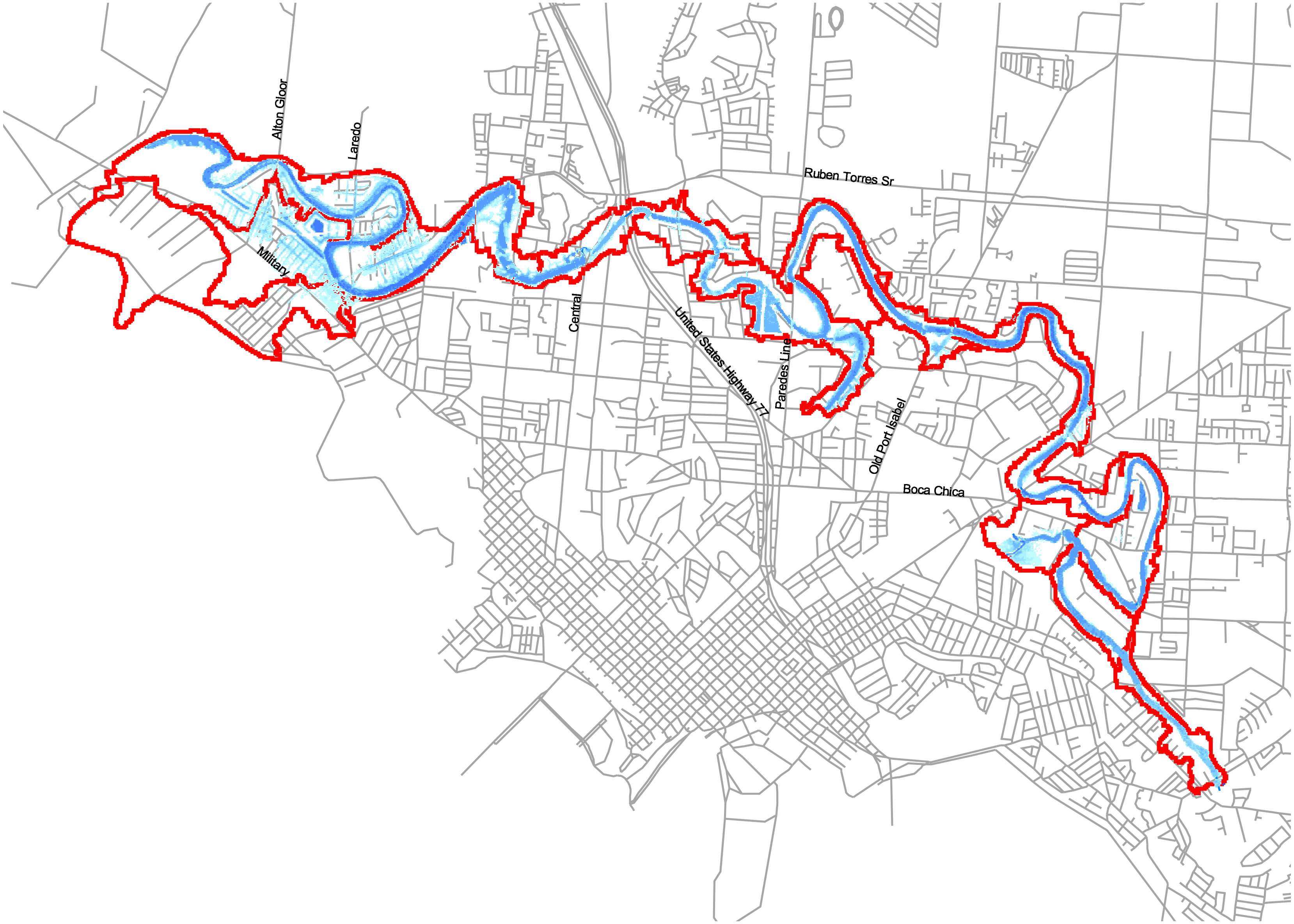
RDLG Watershed
Existing Development
10-Year Floodplain Map

Figure 5-2



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

RDLG
RDLG Watershed
Streets

N

0 0.5 1 Miles

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City of Brownsville, TX

RDLG Watershed
Existing Development
100-Year Floodplain Map

Figure 5-3

Table 5-3. RDLG Existing Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross Section	Profile	Q total (cfs)	WSE (ft)
Alton Gloor	250	84060	10	119	33.74
			100	221	34.53
Laredo	518	75700	10	116	33.69
			100	221	34.47
Golf Course (entering)	1960	67600	10	679	33.12
			100	1105	33.81
Golf Cart Bridge	1500	60700	10	327	32.69
			100	692	32.98
Central Blvd	1530	59700	10	327	32.64
			100	692	32.84
US 83/77	1560	57800	10	327	31.37
			100	692	31.8
Old Alice	1595	56200	10	271	31.08
			100	496	31.42
Paredes Line	1790	49300	10	477	29.31
			100	481	29.33
Old Port Isabel	2158	36150	10	614	28.04
			100	798	28.39
Price Rd	2297	27300	10	614	27.5
			100	798	27.87
Padre Island Highway	2330	24600	10	514	26.19
			100	595	26.3
Boca Chica	2510	15300	10	514	24.33
			100	595	24.66
Billy Mitchell Blvd	2524	13300	10	496	23.72
			100	589	23.88
Acacia Lake Drive	2570	11000	10	496	22.65
			100	589	23.14
Morningside Rd	2958	200	10	484	21.08
			100	746	21.52
Outlet	2966	55	10	484	15.91
			100	708	16.17

Table 5-4. RDLG Flooded Structures for Modeled Design Storms

	Frequency event	
Water Depth (ft)	10-yr	100-yr
0.5 - 1.0	37	61
1 - 1.5	18	37
1.5 - 2.0	8	17
2.0 - 3.0	15	27
3.0 - 5.0	4	6
> 5.0	0	1
Total	82	149

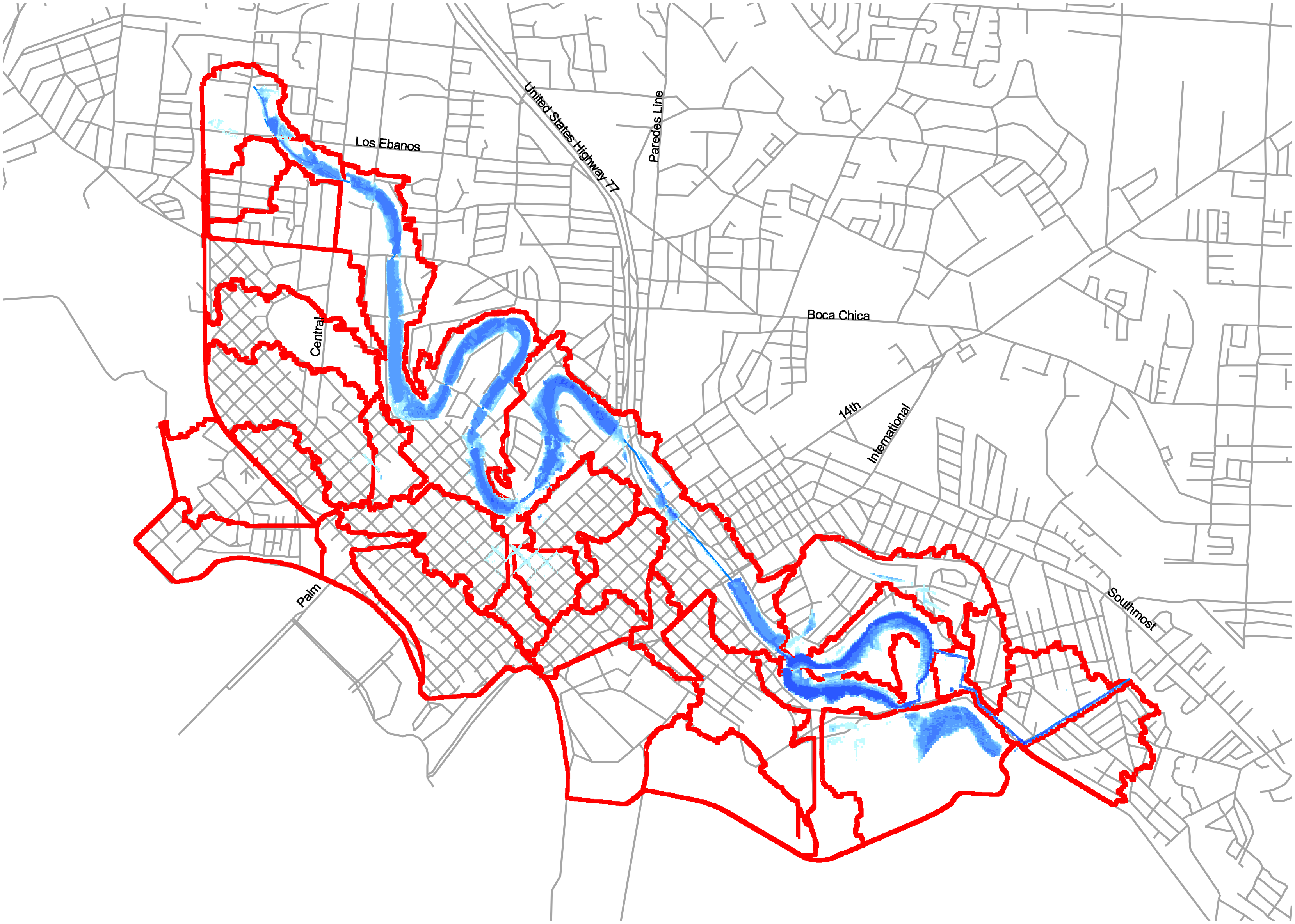
5.2.2 Town Resaca

Town Resaca (TR) flows in a southeasterly direction, for approximately 7.7 river miles through a highly developed portion of Brownsville before draining into North Main Drain. Prior to the Resaca's termination into NMD water can be pumped into the Rio Grande River via the Impala Pump station to minimize the impact of the waters on the already strained NMD. This pump station consists of six pumps with each pump having a capacity of approximately 90 cfs. As was the case with RDLG, only small differences were observed between floodplain delineations for varying storm events throughout the majority of the watershed; although the effect was slightly more pronounced for TR than RDLG (Figures 5-4 and 5-5). In view of the relatively small effect between different intensities, only the results for the 10-yr and 100-yr events are discussed here. The flows and water surface elevations for the 10-yr and 100-yr events may be viewed in Table 5-5. Water surface elevations for all computed frequency events may again be viewed in Appendix B (P-6).

Overall, approximately 13% of the entire TR land area is inundated with water for the 100-yr storm event (nearly 9% for the 10-yr event). The major area of concern in Town Resaca in terms of out of bank flooding is located south of the Gladys Porter Zoo between 3rd St. and 11th St.. This area becomes inundated for all events greater than the 25-yr event. Another smaller area of concern exists exiting the zoo near the intersection of Palm Blvd. and Ringgold St. This low-lying area becomes inundated for all events greater than the 2-yr event, and worsens as you approach the 100-yr storm. There are several homes built in this region that are at high risk of flooding and represent a likely buyout area. Farther downstream below East Ave. lies an additional large area of flooding; however since this area is currently undeveloped, it does not present a threat of damage to structures in this region. Other smaller areas of flooding exist throughout the watershed but are minimal compared to the former locations. Table 5-6 shows the impact of the 10-yr and 100-yr storm events on structures in the watershed.

Table 5-5. TR Existing Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage	Cross Section	Profile	Q Total (cfs)	WSE (ft)
Los Ebanos Blvd.	138	39450	10 100	46 68	31.57 31.85
Central Blvd.	235	37600	10 100	142 246	31.56 31.84
Boca Chica Blvd.	345	35150	10 100	165 330	31.51 31.79
Belthair St.	360	33700	10 100	138 314	29.55 31.33
Calle Retama	661	30300	10 100	138 314	29.52 31.31
Calle Retama	720	25500	10 100	173 342	29.38 31.29
Palm Blvd.	745	24700	10 100	173 342	29.25 31.28
Old Alice	1830	22600	10 100	200.3 330	29.24 31.27
Ringgold St.	1906	20300	10 100	835 1313	28.51 30.5
US 83/77	1993	17200	10 100	806 1239	28.48 30.47
12th St.	2050	15400	10 100	694.4 1096	28.33 30.35
International Blvd	2165	11000	10 100	690 1098	27.36 29.04
East Ave.	3386	4500	10 100	881 1333	25.62 27.55
Impala Drive	3386	2600	10 100	889 1348	25.39 27.38
Calle Milpa Verde Dr.	3494	1700	10 100	889 1348	23.98 25.8
Tulipan	3633	400	10 100	889 1348	21.98 24.28
outlet	3637	213	10 100	1007 1276	16.8 17.35



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

TR
TR Watershed
Streets

N

0 0.5 1 Miles

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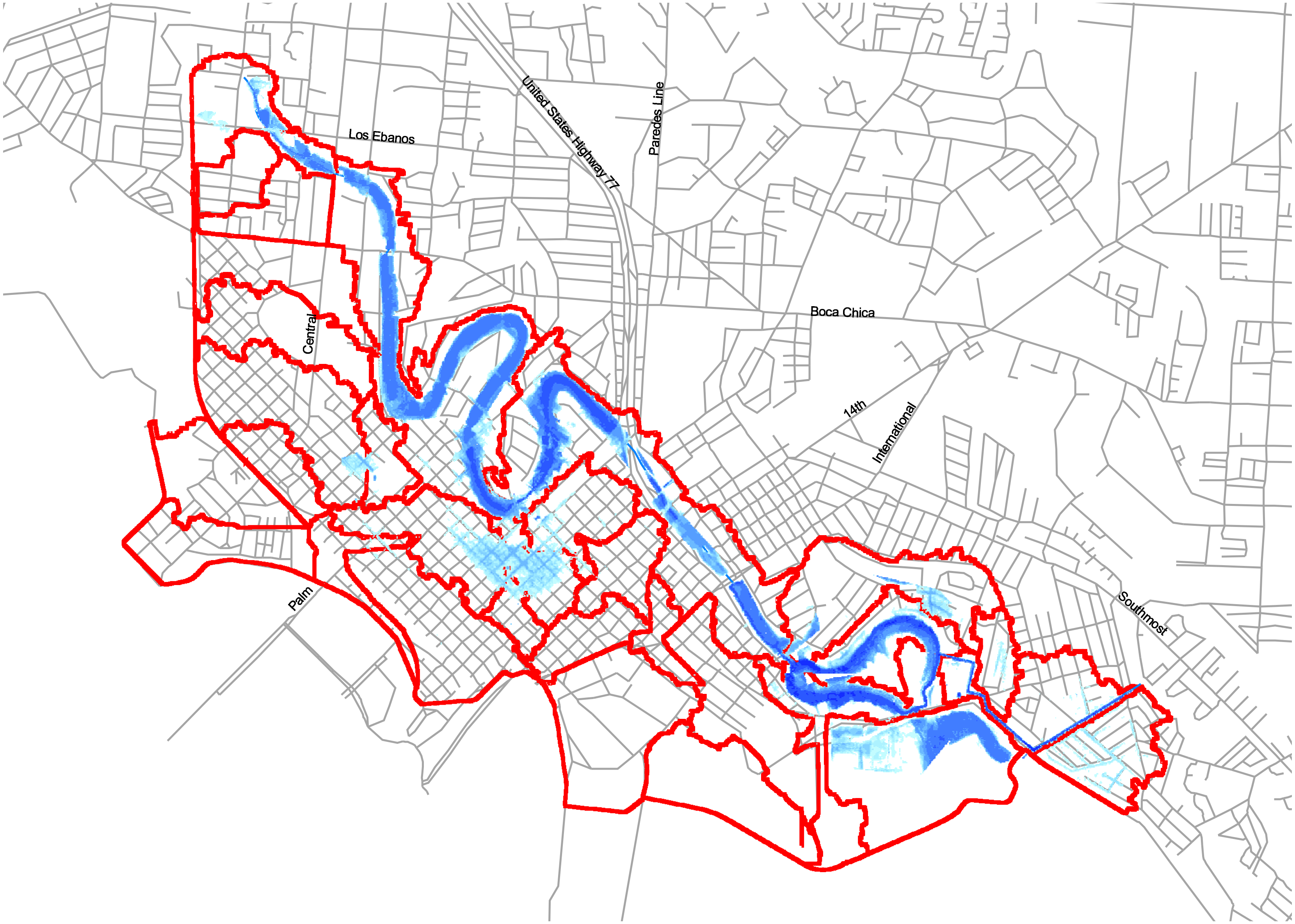
City of Brownsville, TX

TR Watershed
Existing Conditions
10-Year Floodplain Map

Figure 5-4



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

TR
TR Watershed
Streets

N

0 0.5 1 Miles

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City of Brownsville, TX

TR Watershed
Existing Development
100-Year Floodplain Map

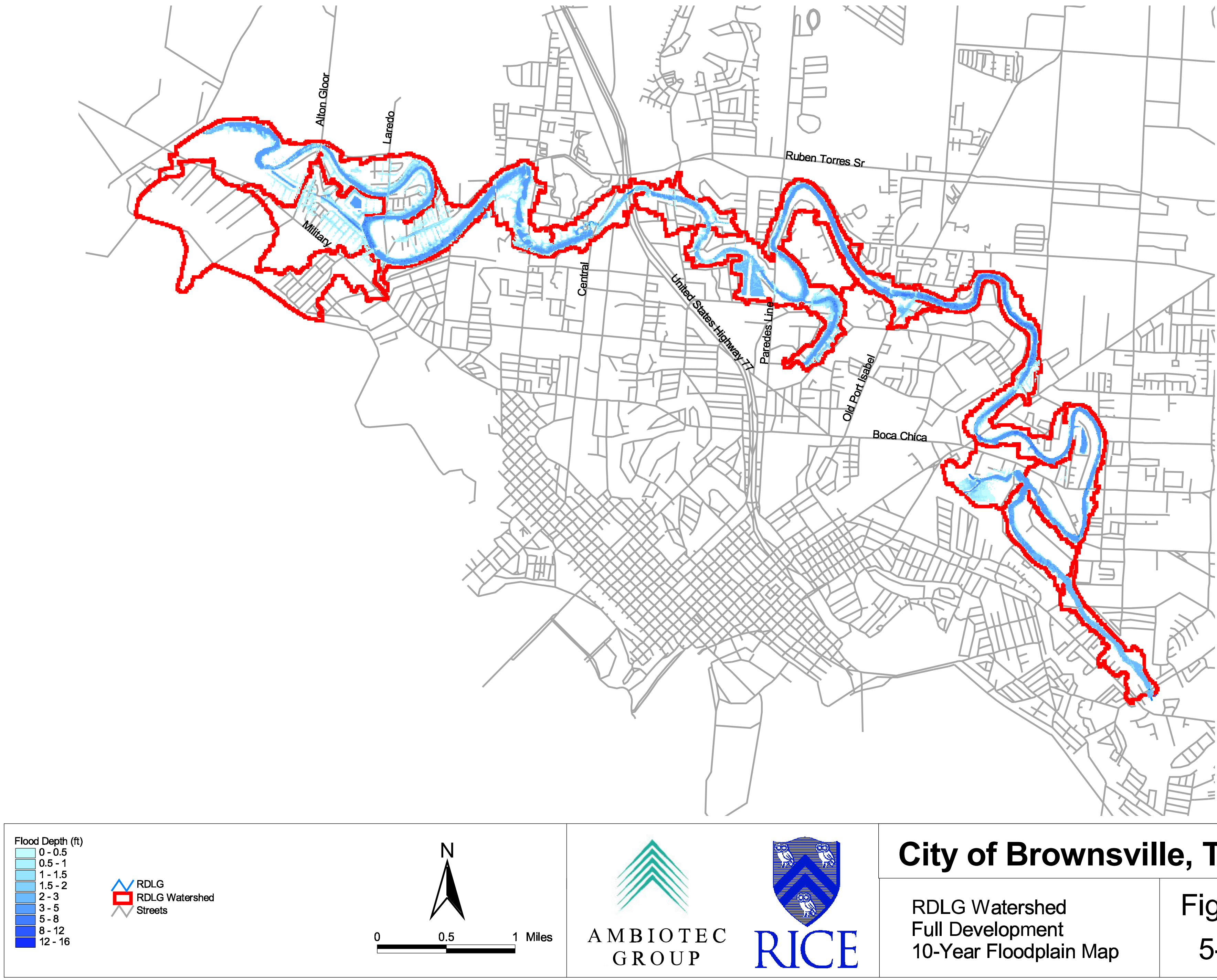
Figure 5-5

Table 5-6. TR Flooded Structures for Modeled Design Storms

	Frequency event	
Water Depth (ft)	10-yr	100-yr
0.5 - 1.0	10	139
1 - 1.5	17	88
1.5 - 2.0	14	17
2.0 - 3.0	27	26
3.0 - 5.0	29	56
> 5.0	9	36
Total	106	362

5.3 Full Development Analysis of Resaca de la Guerra and Town Resaca

To model the effects of a full development land use scheme on the runoff-response of the watershed, the land use map created for the two drainage ditches was used to adjust hydraulic roughness coefficients of the hydrologic and hydraulic resaca models. The 10-yr and 100-yr floodplains for each watershed that resulted from this effort may be viewed in Figures 5-6, 5-7, 5-8, and 5-9. Water surface elevations for the 2-yr through 100-yr events may be viewed in Appendix B (P-7 and P-8). Generally this adjustment of roughness coefficients resulted in an increase of peak flow rates, water surface elevations, and the total out of bank flooding volume (Tables 5-7, 5-8, 5-9 and 5-10). However, when comparing these tables to those for the existing condition scenario it may be observed that despite a large increase of flow, the increase in predicted water surface elevations is minimal. This occurs due to the characteristically high banks of the resacas and consequently the large capacity for excess stormwater that they hold. Tables 5-9 and 5-10 illustrate the increase in out of bank flood volume as well as floodplain area for the existing verses full development scenarios.



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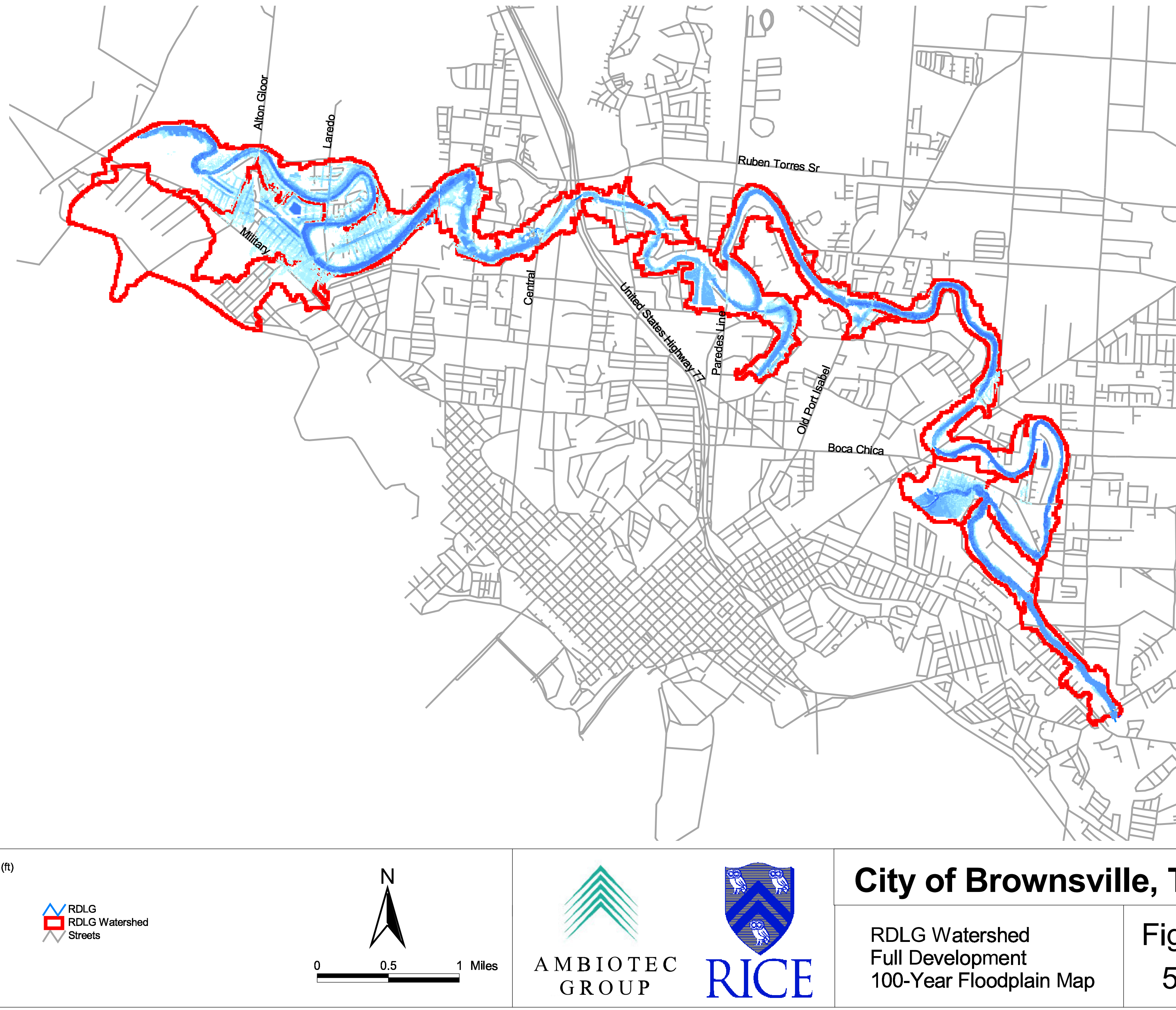
City of Brownsville, TX

RDLG Watershed
Full Development
10-Year Floodplain Map

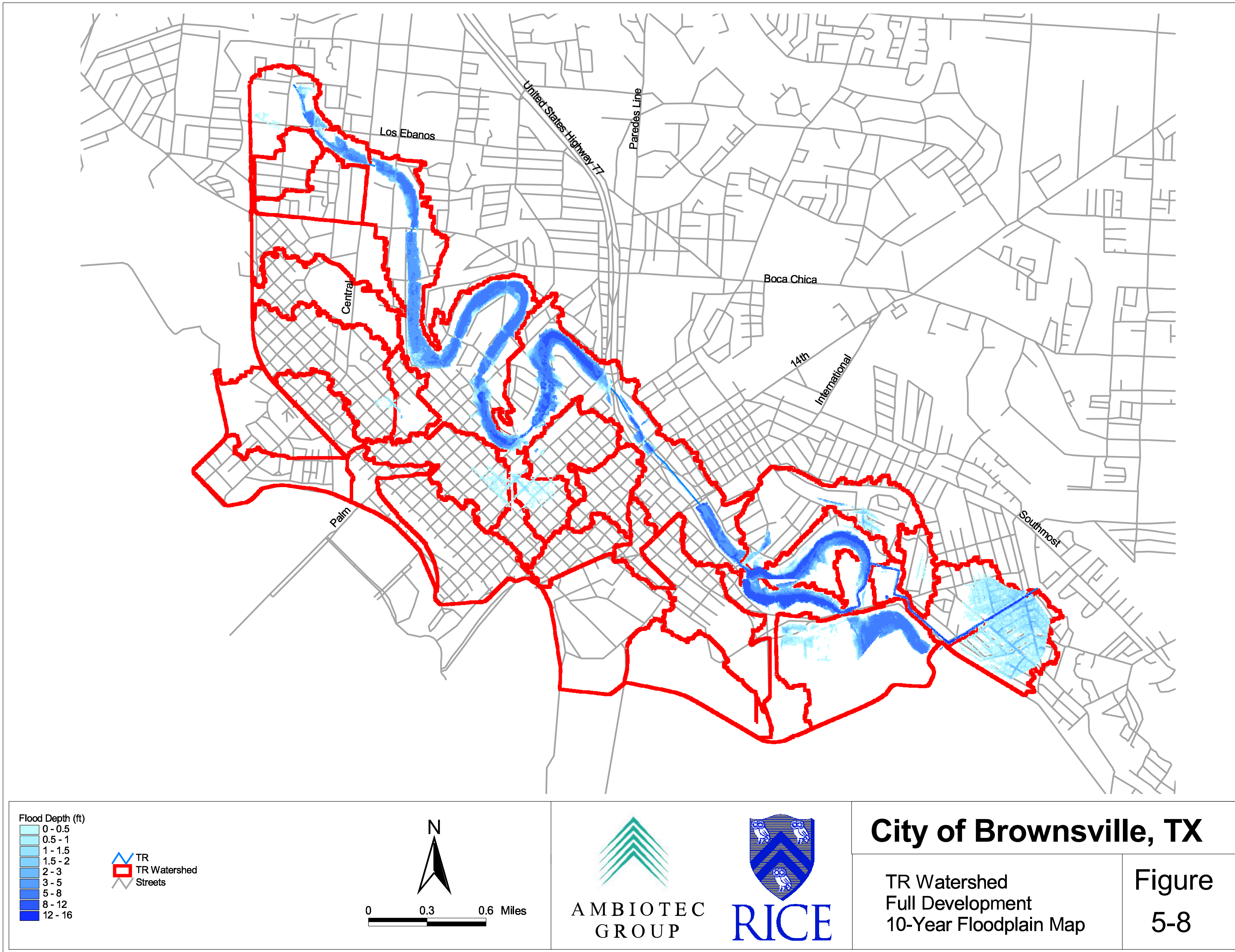
Figure 5-6



City of Brownsville, TX

RDLG Watershed
Full Development
100-Year Floodplain Map

Figure 5-7



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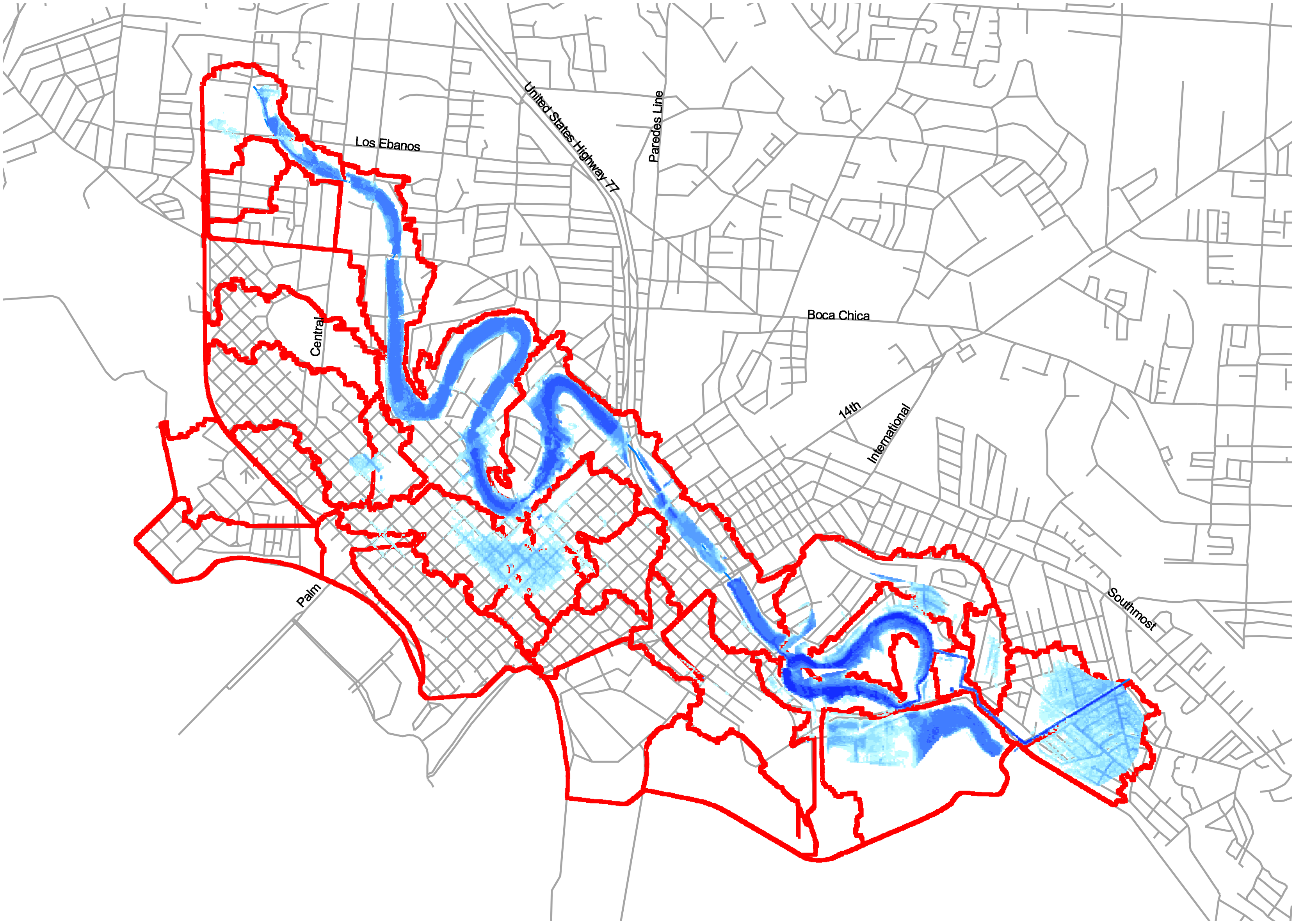
City of Brownsville, TX

TR Watershed
Full Development
10-Year Floodplain Map

Figure
5-8



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

TR
TR Watershed
Streets

N

0 0.3 0.6 Miles

AMBIOTEC GROUP

RICE

City of Brownsville, TX

TR Watershed
Full Development
100-Year Floodplain Map

Figure
5-9

Table 5-7. RDLG Future Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross Section	Profile	Q Total (cfs)	WSE (ft)
Alton Gloor	250	84060	10	205	33.98
			100	357	34.87
Laredo	518	75700	10	209	33.85
			100	372	34.7
Golf Course (entering)	1960	67600	10	735	33.36
			100	1176	34.13
Golf Cart Bridge	1500	60700	10	457	32.72
			100	883	33.17
Central Blvd	1530	59700	10	457	32.63
			100	883	33
US 83/77	1560	57800	10	457	31.54
			100	883	32.03
Old Alice	1595	56200	10	343	31.22
			100	626	31.56
Paredes Line	1790	49300	10	450	29.23
			100	482	29.36
Old Port Isabel	2158	36150	10	817	28.45
			100	1014	28.77
Price Rd	2297	27300	10	817	27.89
			100	1014	28.11
Padre Island Highway	2330	24600	10	551	26.23
			100	657	26.37
Boca Chica	2510	15300	10	551	24.45
			100	657	25.28
Billy Mitchell Blvd	2524	13300	10	538	23.81
			100	650	24.92
Acacia Lake Drive	2570	11000	10	538	23.09
			100	650	24.85
Morningside Rd	2958	200	10	598	22.09
			100	905	24.29
Outlet	2966	55	10	550	21.99
			100	877	24.22

Table 5-8. TR Future Conditions Flows (Q Total) and Water Surface Elevations (WSE)

Location	Drainage Area (acres)	Cross Section	Profile	Q Total (cfs)	WSE (ft)
Los Ebanos Blvd.	138	39450	10 100	47 70	31.48 32.07
Central Blvd.	235	37600	10 100	159 268	31.61 32.03
Boca Chica Blvd.	345	35150	10 100	213 445	31.6 31.95
Belthair St.	360	33700	10 100	169 416	30.28 31.4
Calle Retama	661	30300	10 100	169 416	30.27 31.38
Calle Retama	720	25500	10 100	196 408	30.12 31.37
Palm Blvd.	745	24700	10 100	196 408	29.95 31.35
Old Alice	1830	22600	10 100	204 419	29.94 31.34
Ringgold St.	1906	20300	10 100	841 1322	29.37 30.65
US 83/77	1993	17200	10 100	814 1255	29.34 30.62
12th St.	2050	15400	10 100	714 1119	29.25 30.5
International Blvd	2165	11000	10 100	714 1132	28.24 29.51
East Ave.	3386	4500	10 100	902 1434	27.15 27.87
Impala Drive	3386	2600	10 100	914 1453	27.05 27.71
Calle Milpa Verde Dr.	3494	1700	10 100	914 1453	26.55 27.02
Tulipan	3633	400	10 100	914 1453	26.46 26.94
Outlet	3637	213	10 100	1007 1416	26.42 26.9

Table 5-9. Total out of bank flooding volumes for the 10-yr and 100-yr Storm Events in RDLG

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	678	675	926	842
Full Development	762	725	1196	970
% Increase	11%	7%	23%	13%

Table 5-10. Total out of bank flooding volumes for the 10-yr and 100-yr Storm Events in TR

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	567	319	1032	494
Full Development	840	456	1245	617
% Increase	33%	30%	17%	20%

It may also be observed that while the peak flow rates increased dramatically in certain regions corresponding with large areas of land use change, the flows recover moving downstream. This occurs because of the large storage capacity within the resacas and the fact that flows out of each separate pool are hydraulically controlled. Furthermore, this illustrates the large storage capacity that the resacas have to absorb excess flow without causing a striking impact on the floodplain.

The floodplain delineations for RDLG and TR under the future land use scenario reveal the same problem areas as what was described for the existing condition delineations. The major difference however is that the depth of flooding that occurs in these areas is now slightly greater. In the RDLG watershed there was a very slight increase in the extent of the floodplain for each frequency storm event but the majority of the out of banks volume increase, while minimal, was caused by an increase of water depth. It may also be noted that once again the floodplain delineation for each of the design storms does not differ by very much from one to the next in terms of total inundated area. This further indicates that the problem areas within RDLG are due to the fact that they are significantly lower than the surrounding area.

For the TR watershed the same problem areas exist except the downstream area before the outlet is now inundated with water for all modeled frequency events. Street flooding in this region is as deep as 3.5-ft with overland flooding approaching depths of 1.5-ft for the 100-yr event. With the exception of this area, only small differences in the total extent of the predicted floodplains were observed between the existing and future land use conditions.

The impact that the delineated floodplains had on structures in the region may be viewed in Tables 5-11 and 5-12. These counts were made by assessing the number of structures already existing in the watershed that would further be influenced by the full development floodplain. Unlike the floodplains in CCDD1 and NMD, the floodplains around the resacas are very narrow and cover an area that is already more or less developed (mostly with residential neighborhoods) and not likely to change. Therefore, because the floodplains do not extend to areas far from the banks of the resaca where land use change is likely, it would not make sense to count the number of structures in the floodplain assuming a full development density of 2.5 structures per acre. Instead, the additional number of structures impacted by the full development floodplain relative to the existing development floodplain was determined by the existing housing density.

In the RDLG watershed it may be observed that a large increase in impacted structures is experienced for the 100-yr event. The majority of this difference is due to the increase in floodplain area surrounding the second tributary in the four corners area (Boca Chica and International Blvd.).

Table 5-11. Number of structures within the 10-yr and 100-yr floodplains for RDLG for the future development scenario

	Existing Development		Future Development			
Water Depth (ft)	**10-Year**	**100-Year**	**10-Year**	**% Increase**	**100-Year**	**% Increase**
0.5 - 1.0	37	61	45	18%	105	42%
1 - 1.5	18	37	17	-6%	70	47%
1.5 - 2.0	8	17	17	53%	26	35%
2.0 - 3.0	15	27	15	0%	31	13%
3.0 - 5.0	4	6	7	43%	17	65%
> 5.0	0	1	1	100%	1	0%
Total	82	149	102	20%	250	40%

In the TR watershed a large relative difference was observed in the total amount of flooded structures for the full development scenarios verses that of the existing. The 10-yr floodplain impacted an additional 309 structures and the 100-yr an additional 537. This large change in flooded structures is predominantly due to the increase in area of the floodplain in the downstream portion of the watershed near the outlet into NMD.

Table 5-12. Number of structures within the 10-yr and 100-yr floodplains for TR for the future development scenario

	Existing Development		Future Development			
Water Depth (ft)	**10-Year**	**100-Year**	**10-Year**	**% Increase**	**100-Year**	**% Increase**
0.5 - 1.0	10	139	206	95%	352	61%
1 - 1.5	17	88	99	83%	296	70%
1.5 - 2.0	14	17	19	26%	114	85%
2.0 - 3.0	27	26	29	7%	41	37%
3.0 - 5.0	29	56	48	40%	57	2%
> 5.0	9	36	14	36%	39	8%
Total	106	362	415	75%	899	60%

6.0 RECOMMENDED FLOOD PROTECTION PLAN

This section outlines the elements of the proposed Flood Protection Plan for the study area. The goal of the plan is to reduce the effect of flooding on the existing and future development in a cost effective manner. The key element of the Flood Protection Plan is a Capital Improvement Plan (CIP) that describes the type and implementation schedule of both structural and non-structural flood control projects that can meet the plan goals in a cost effective manner.

The CIP is a flexible document. It will be subject to change in response to future development patterns, budgetary and regulatory constraints, and political priorities. Furthermore, preliminary and final engineering design phases must be completed prior to finalizing the details of each project in the CIP.

The Flood Protection Plan was developed by selecting a number of candidate flood control options; testing the efficacy of each option by running the H&H models to determine the reduction in water surface elevations and flood damages versus the cost of implementing the option; ranking the alternatives by cost effectiveness; and selecting and giving the highest priorities to those projects that resulted in the greatest flood reduction for a given increment in cost. The selected projects were then organized into a 20-yr CIP consisting of a sequence of four 5-yr CIP plans. The total capital cost for the proposed CIP totals $182 million and results in reduced flood damages that exceed $300 million over the 20-year planning period.

The remainder of this section presents the modeling results for each of the selected alternatives for each of the four watersheds, including the effect of each of the options on the water surface elevations and number of impacted structures; the costs and benefits of the selected alternatives; and project summaries of the proposed 20-yr CIP together with potential funding options.

6.1 Structural versus Non-Structural Improvements

A variety of structural and non-structural methods for reducing flooding were analyzed for the applicability to the specific problem areas identified for each watershed. Generally, methods that use engineered structures to modify the flood runoff are classified as structural methods while non-structural methods serve to adjust the use of flood-prone lands and restrict the timing and amount of runoff as development in a region takes place.

6.1.1 Structural Improvements

The traditional approach to flood control since the early 1900s relied upon major structural alterations to channels and the building of dams and reservoirs (Bedient and Huber, 2001). Since then the number of structural flood control options has expanded to include such measures as the construction of detention/retention ponds, and the building of levees or dikes. Among the structural methods considered for this study are the construction of detention ponds (especially multi-use detention ponds), diversions, channel modifications, levees, and bridge/culvert improvements. While such structural methods have proven effective over time in reducing floodwaters during storm events, there is often a significant capital and operations expense associated with their implementation. Furthermore, it is important to stress that such structural improvements become less effective as the peak flows generated by a storm event increase and surpass the capacity that the structure was designed to accommodate. Therefore, any changes in land-use patterns within the watershed that alter the runoff response of the watershed to produce larger flows will change the effectiveness of the given structural improvement. For this reason, such structural flood control techniques often work best in conjunction with the use of non-structural techniques.

6.1.2 Non-Structural Improvements

Non-structural flood control alternatives are a popular mitigation method due to their relatively inexpensive nature and for their effectiveness. Non-structural methods include such options as runoff controls/impact fees on future developments, buy-outs of flood prone areas, flood alert systems, development of regulatory ordinances, and administrative reorganizations. While such methods are not very effective in mitigating a pre-existing flooding problem, they are extremely effective in preventing a problem from worsening or occurring at all in areas that have not yet reached their full development level. With the exception of buy-outs of flood prone areas, the majority of the non-structural methods, in particular, the development of regulatory ordinances, runoff controls/impact fees, and administrative reorganizations, will apply across all watersheds. However, because all the watersheds are close to full development levels with the exception of CCDD1, regulatory controls and ordinances will have a greater applicability for CCDD1. The recommended runoff controls will simply state that the runoff from a given site shall be equal to or less than the existing, non-developed flows. Furthermore, any future developments must not result in an increase of the existing, pre-developed water surface elevations anywhere within the watershed.

6.2 North Main Drain

Existing flood control measures in the North Main Drain include the Impala Pump Station, which diverts up to 540 cfs from the Town Resaca/North Main Drain confluence; a levee/detention pond near the Ruiz Street area; and a levee along the northern bank of the airport. Nevertheless, the North Main Drain still experiences significant flooding problems throughout its entire drainage system. The simulated 100-yr floodplain results in the inundation of over 2,300 acres, or approximately 40% of the entire watershed, under both existing and future development conditions (Table 4-13).

Options for reducing flooding in the NMD are limited because the watershed is nearly fully developed, and because the right-of-way available to widen the ditch is severely constrained throughout the reach; and while system-wide runoff control/impact fee policies for new development are an important and effective component of the overall plan for the study area, the relevance to the NMD is marginal since this watershed is already close to its ultimate level of development.

Ten structural alternatives were considered for the North Main Drain including detention ponds, diversions, channel modifications (only in locations where the right-of-way constraints are not too limiting), levees, and hydraulic structure improvements. In order to facilitate the estimation of the benefits and costs of the alternatives, they were grouped into five sequential options (A, B, C-1, C-2, and C-3); with each subsequent option built on the previous set of alternatives by including an additional group of alternatives. The five specific alternatives include:

- Option A: Five detention ponds ranging in storage capacity from 60 ac-ft near Coria Road up to almost 3,800 ac-ft at the airport. Also, the construction of a levee on the southern side of the airport.
- Option B: Option A plus channel improvements from Hwy 77/83 to the confluence with Resaca de la Guerra
- Option C-1: Option B plus diversion of 1,000 cfs west of flows greater than 2-yr return period at the confluence with Hwy 77/83 to the Rio Grande
- Option C-2: Option B plus diversion of 1,500 cfs west of all flows at the confluence with Hwy 77/83 to the Rio Grande
- Option C-3: Option C-2 plus a pump station with an additional capacity of 2,500 cfs at the Four-Corners area to divert flow into Resaca de la Guerra

Due to the similarity of the existing and full development base case scenarios, options for NMD were only modeled for the full development conditions.

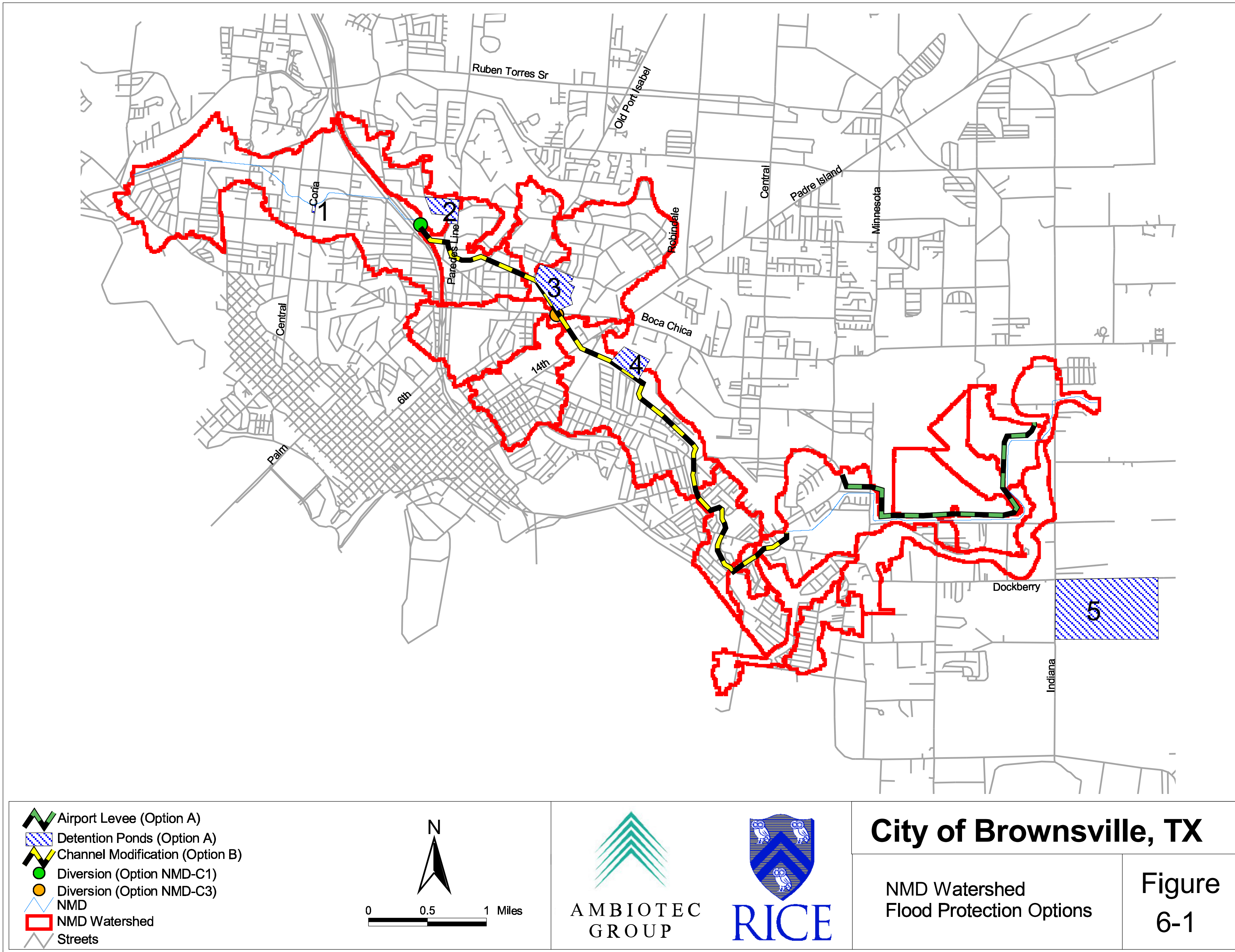
6.2.1 Option A

Option A includes the construction of five detention ponds diverting water from the drainage ditch. The amount of available land limits the number and size of the proposed detention ponds for the North Main Drain watershed. Four of the proposed ponds would be offline detention ponds with a hydraulic structure, such as a weir, governing the amount of flow entering the pond. Additionally, they would be designed as multiple purpose ponds that would provide recreational benefits in addition to flood protection. The fifth pond located near the southeast corner of the airport is outside of the watershed area and thus it will be necessary to install a pump station to divert water to this site. Inline detention ponds are not planned for the watershed because they are not effective in reducing the volume of water in the channel, which is needed to reduce the floodplain. In addition, a levee over 2.5 miles long would be constructed around the southern side of the airport. This levee would be constructed to a height of approximately 24-ft (about 1-ft over the 100-yr WSE) and would be about 10-ft wide with 2:1 side slopes. The detention ponds for the NMD watershed are listed in Table 6-1 and shown in Figure 6-1.

Table 6-1. Proposed Detention Ponds in NMD

Map Point	Location	Acreage (acres)	Capacity (ac-ft)
1	Near Coria Rd and NMD	6.5	59
2	Near East Price Rd	12	94
3	Old Port Isabel Rd and NMD	45	345
4	Downstream from International Blvd	20	162
5	Near southeast corner of the airport	300	3771

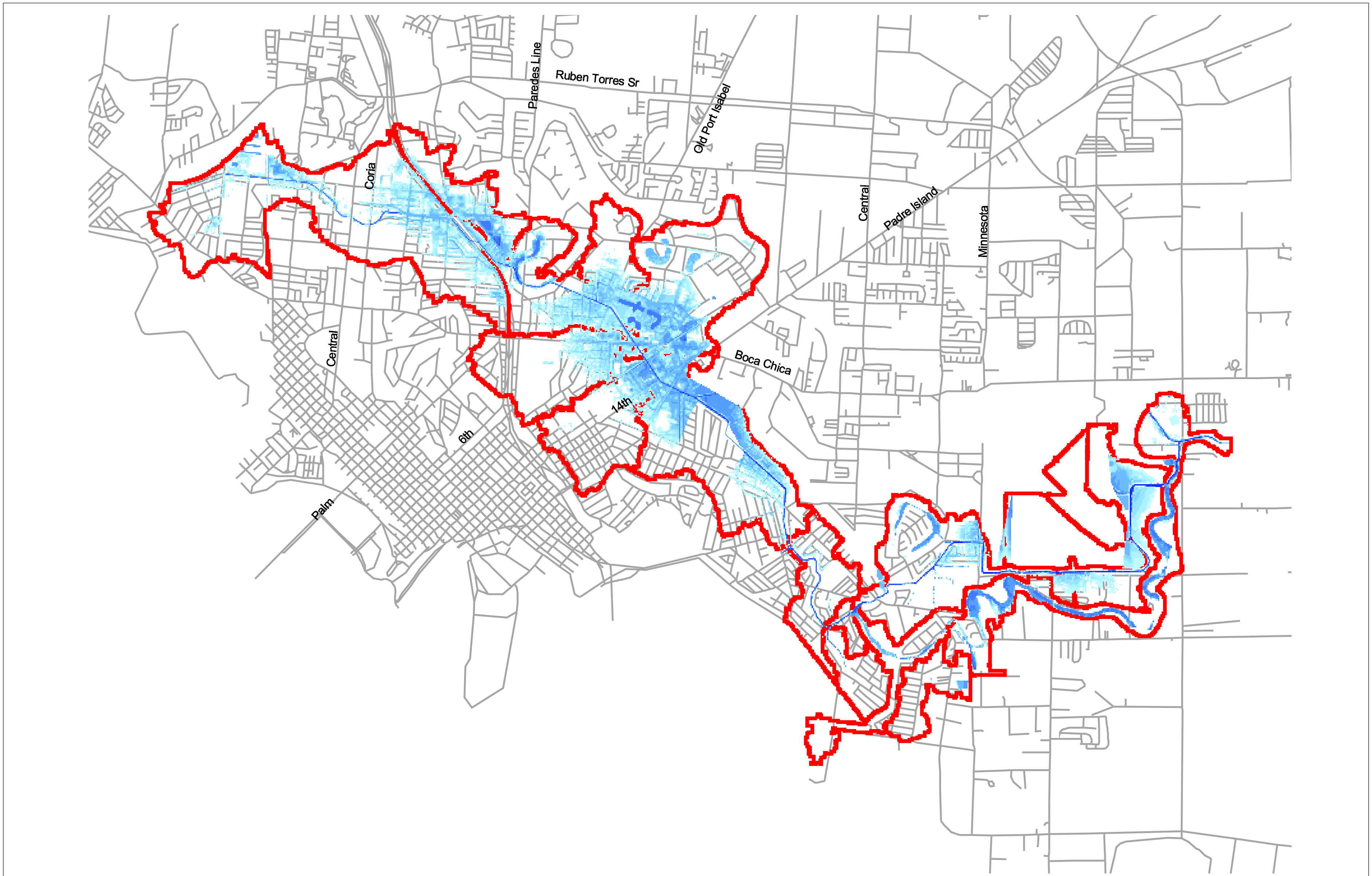
The effects of the simulated implementation of Option A may be viewed in Tables 6-2, 6-3, 6-4 and 6-5. The implementation of Option A reduces the overland coverage of the 100-year floodplain by approximately 24 percent and the out of bank volume by approximately 34 percent for the existing/full development conditions relative to the no action scenario. This reduction in volume and area of the floodplain resulted in a drop in the number of impacted structures from 3191 to 2382. The overland runoff from the majority of the airport has been redirected into the airport detention pond, bypassing the ditch entirely. It is important to note once again that the topography plays an important role in floodplain reduction. So, while the implementation of Option A reduced the extent of the floodplain by only 24% (most of which was from the airport), the floodplain was reduced by 1.3 ft near the outlet for the 100-year event in terms of the water surface elevations. The 10-yr and 100-yr floodplains for Option A may be viewed in Figures 6-2 and 6-3. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.



City of Brownsville, TX

NMD Watershed
Flood Protection Options

Figure
6-1



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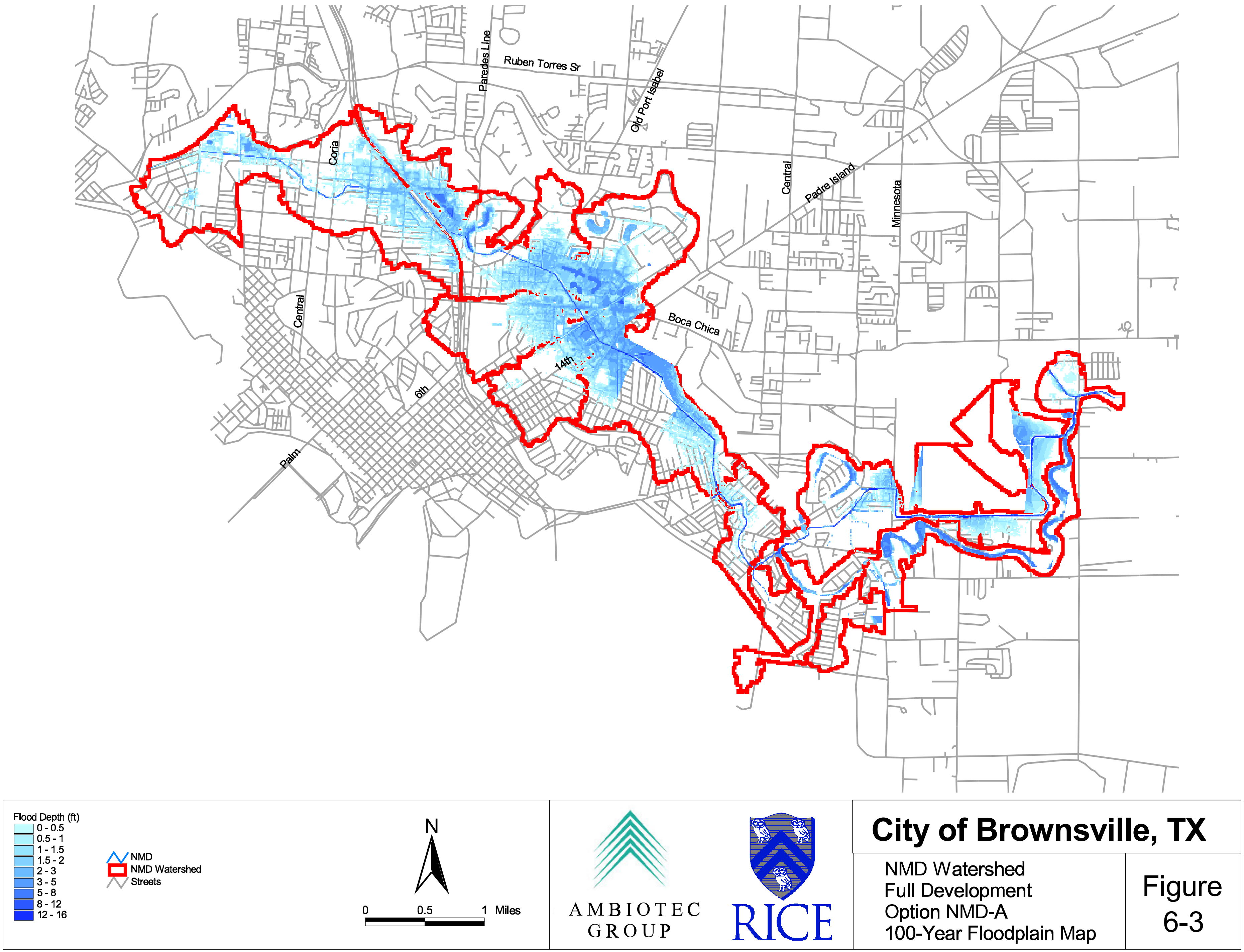
City of Brownsville, TX

NMD Watershed
Full Development
Option NMD-A
10-Year Floodplain Map

Figure
6-2



City of Brownsville, TX

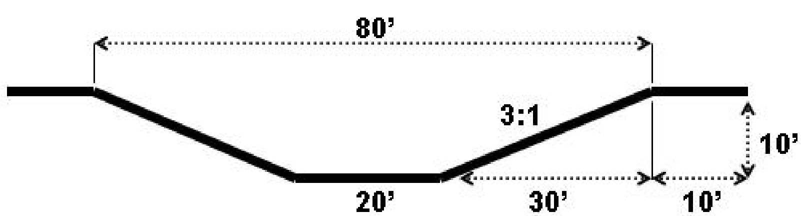
NMD Watershed
Full Development
Option NMD-A
100-Year Floodplain Map

Figure 6-3

6.2.2 Option B

Option B includes the construction of all detention ponds and the airport levee from Option A in addition to channel improvements to the North Main Drain ditch from Highway 77/83 to the Resaca de la Guerra confluence as seen in Figures 6-1. The channel improvement includes widening the channel assuming a right-of-way of 100 feet, and concrete lining the channel to reduce the energy loss in friction between the channel and water flowing in the channel. The specifications of the channel modification are shown in Figure 6-4.

Figure 6-4. Channel modification specifications



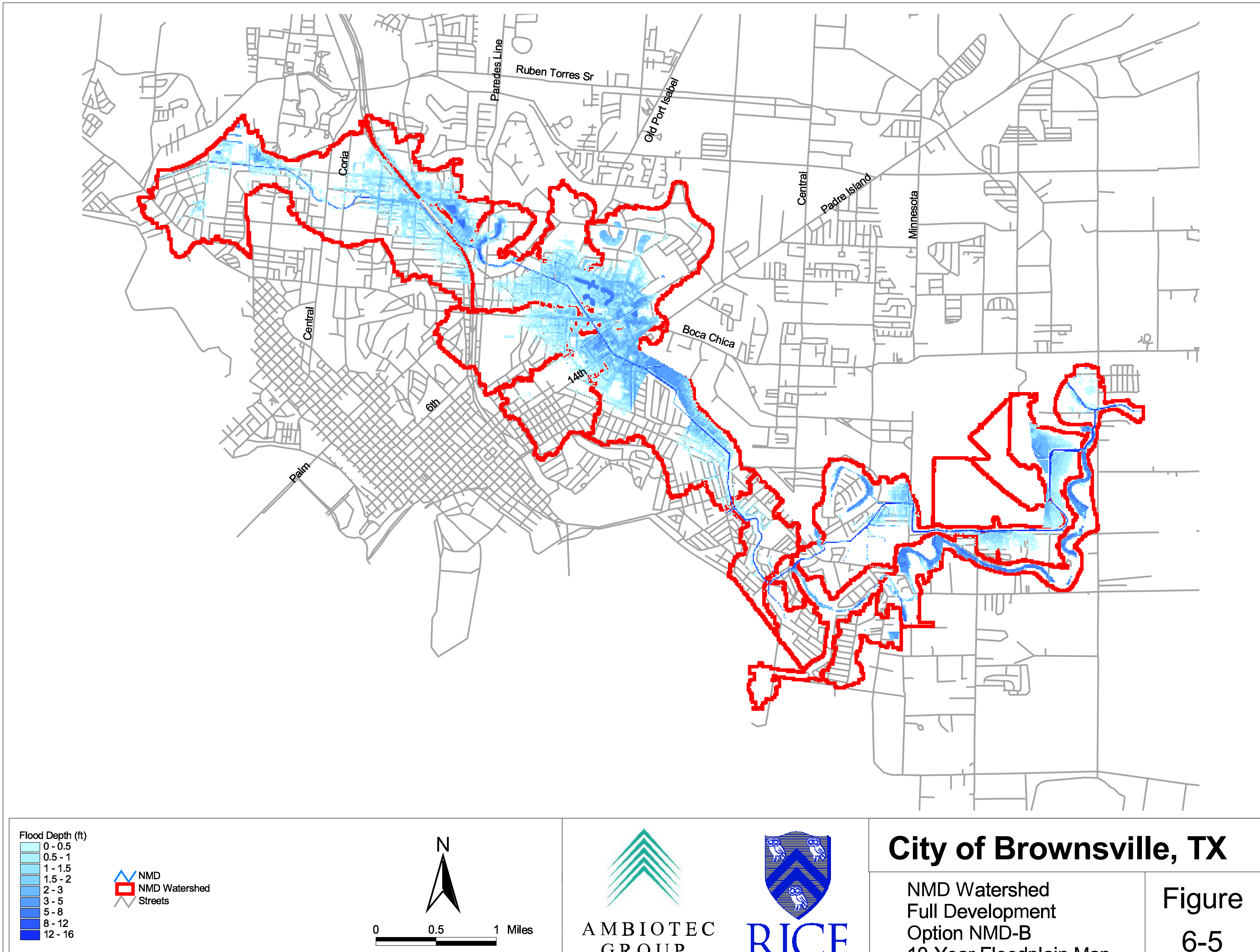
Option B reduces the 100-year floodplain by 22 percent and the out of banks volume by 33 percent compared to the full development no action scenario. The number of impacted structures from Option A to Option B actually increases from 2382 to 2467 structures. This occurs due to a backup of water at the end of the proposed channelization segment at the confluence of RDLG with NMD. At this location the problem is worsened as the channel changes from a relatively wider, concrete lined channel to a narrower, grass-lined channel, which consequently conveys water much more slowly. However, this only occurs for the larger storm events. For events equal to or less than the 25-yr, the number of impacted structures decreased thus making this a more effective option for those smaller events. Tables 6-3 and 6-5 confirm these results. Based on these results it may be concluded that any efforts to channelize NMD should be carried out for the length of the ditch. Figures 6-5 and 6-6 show the 10-yr and 100-yr floodplains resulting after implementation of this option. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

6.2.3 Option C

Because of the various limitations previously discussed (e.g., limited ROW available for expansion of drainage ditch; high level of existing development without runoff controls), it is not possible to significantly reduce flooding in the NMD without recourse to major diversions outside of the watershed. Three diversion options were considered (Options C-1, C-2 and C-3). The first two divert flows from the NMD into the Rio Grande, and the third option diverts flows to RDLG. Although the water surface elevation profiles within the watershed were simulated, a complete simulation of the impacts of the diversion on the receiving watershed was not possible within the current project scope without significant reconfiguration of the models. A more complete evaluation of the feasibility and impacts of Options C-1, C-2 ad C-3 on the receiving basins will need to be completed as part of the preliminary engineering phase of the CIP.

6.2.3.1 Option C-1:

Option C-1 includes Option B with an added diversion of 1,000 cfs at Highway 77/83 to redirect all flow above the 2-year storm flow to the Rio Grande. Water in this scenario would be diverted westward and ultimately pumped into the Rio Grande River. Option C-1 reduces the 100-year floodplain by 30 percent and the out of banks volume by 42 percent compared to the full development no action conditions. This reduction in floodplain area resulted in a reduction of flooded structures from 3191 to 2075 relative to the no-action scenario. Furthermore this is a reduction of an additional 307 structures relative to Option A. Floodplains for this option may be viewed in Figures 6-7 and 6-8. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.



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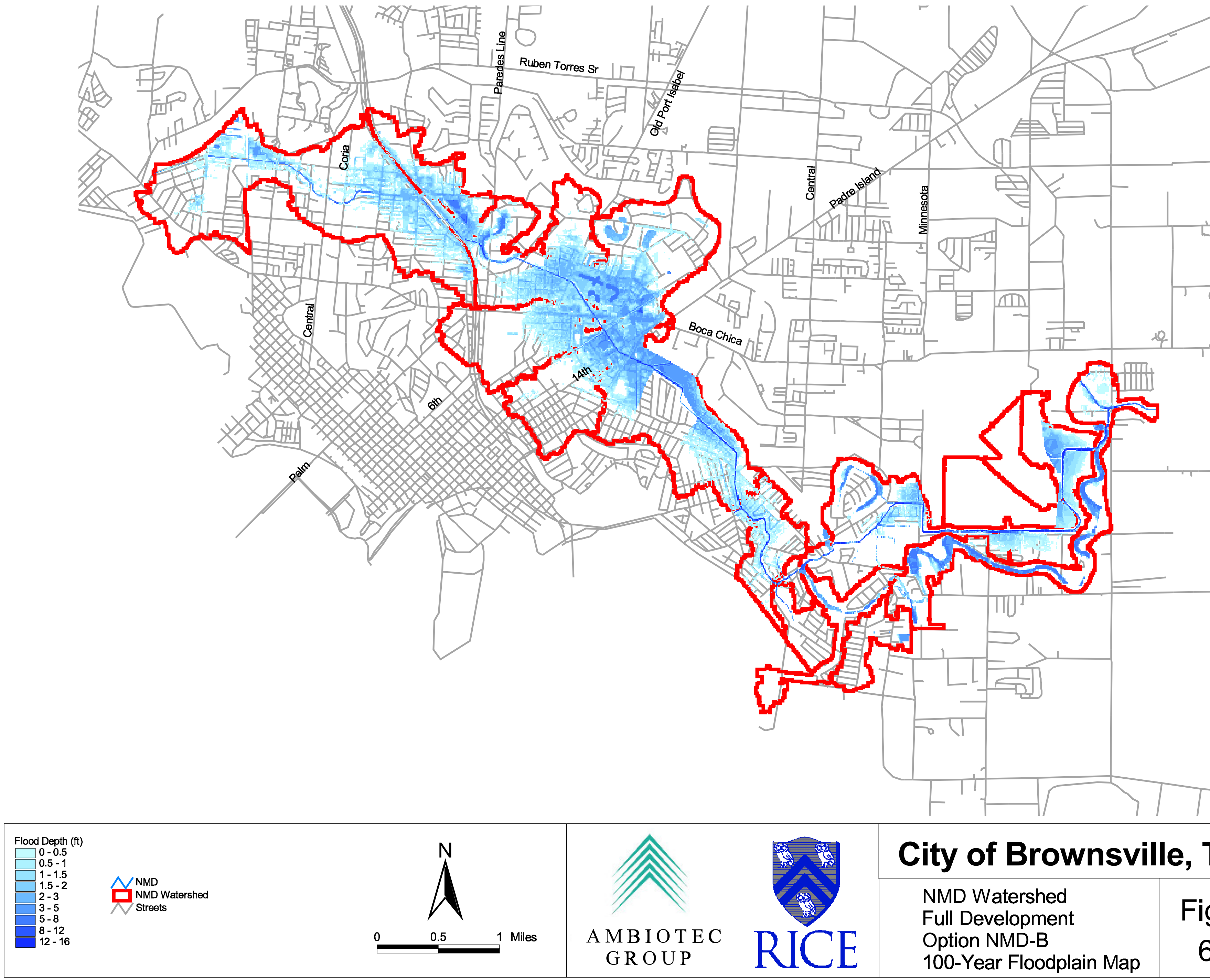
NMD Watershed
Full Development
Option NMD-B
10-Year Floodplain Map

Figure
6-5



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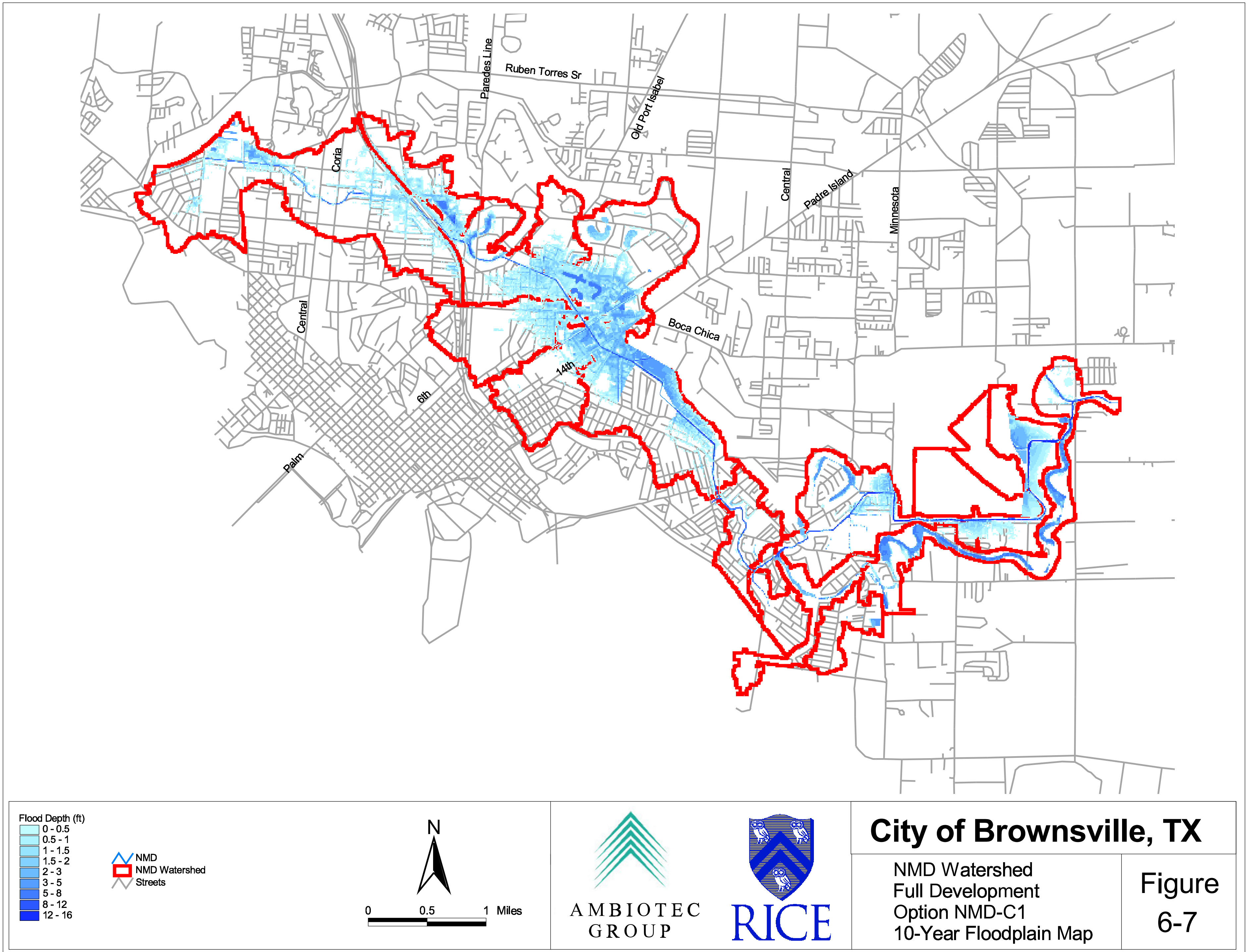
NMD Watershed
Full Development
Option NMD-B
100-Year Floodplain Map

Figure 6-6



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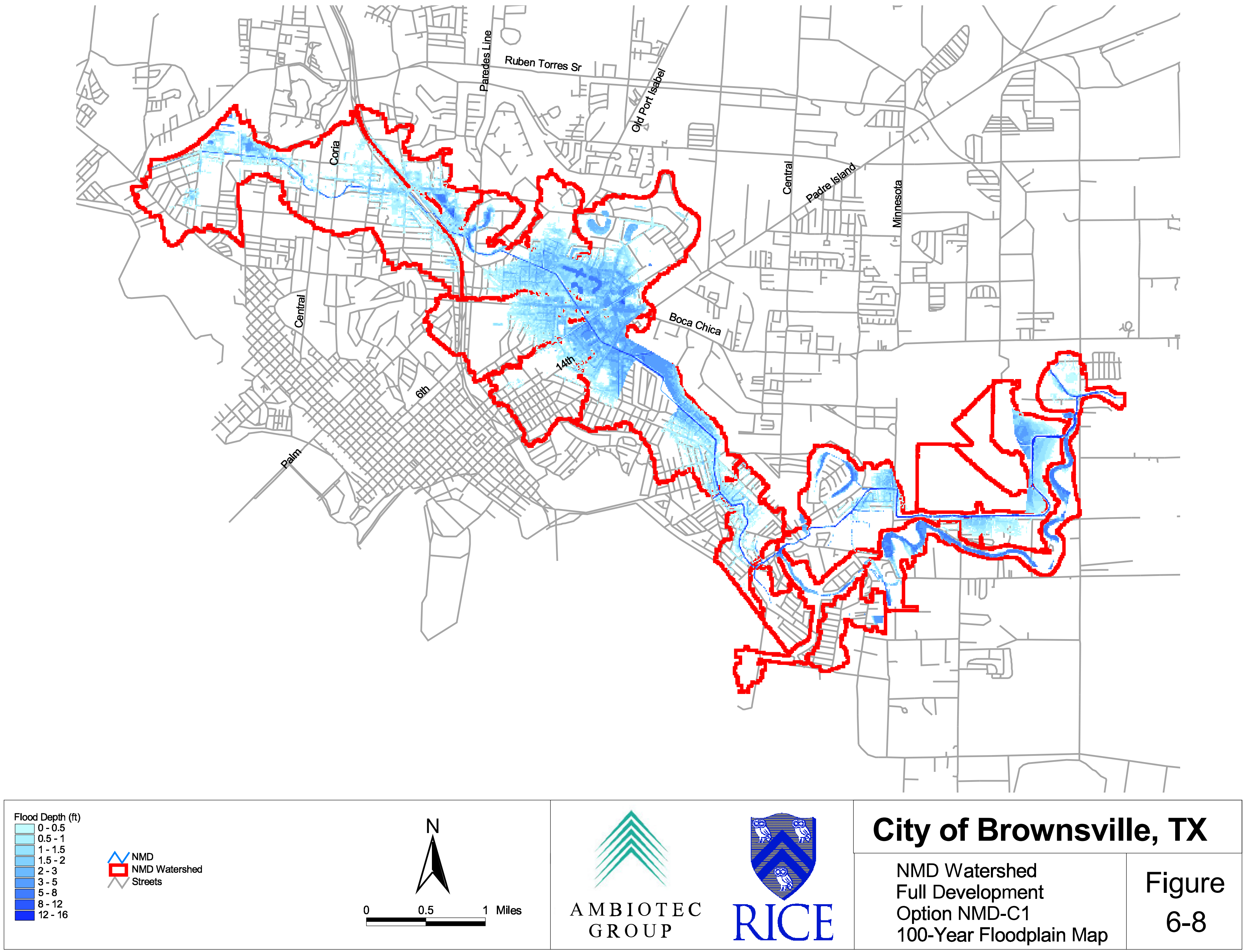
NMD Watershed
Full Development
Option NMD-C1
10-Year Floodplain Map

Figure 6-7



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City of Brownsville, TX

NMD Watershed
Full Development
Option NMD-C1
100-Year Floodplain Map

Figure 6-8

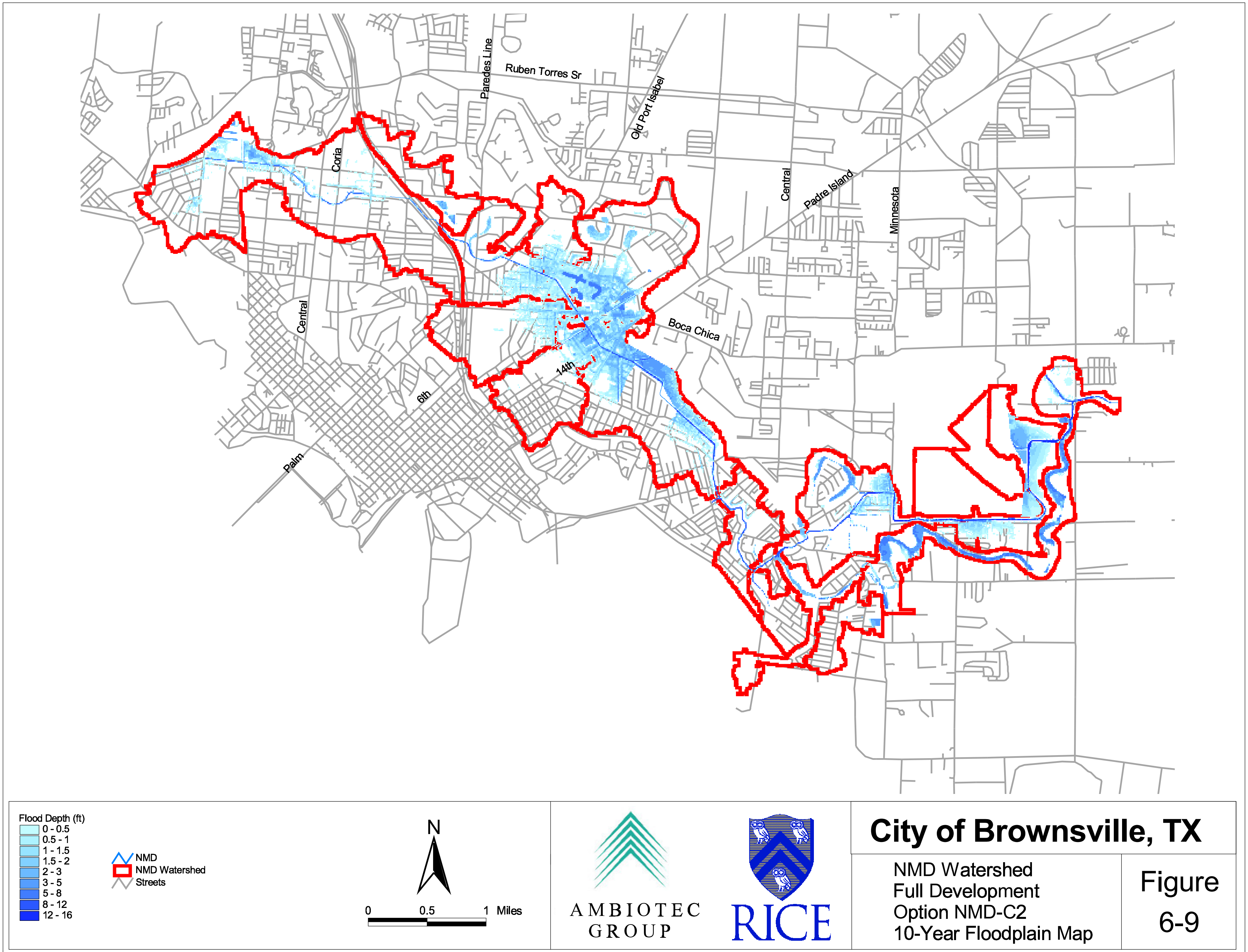
6.2.3.2 Option C-2

Option C-2 is equivalent to Option C-1, but increases the diversion capacity by an additional 1,500 cfs (total capacity of 2,500 cfs) to divert flows upstream of Highway 77/83 to the Rio Grande River. Option C-2 reduces the 100-year floodplain by 39 percent and the out of banks volume by 58 percent compared to the full development conditions. This represents an additional 13% reduction in floodplain area and a 12% reduction in out of banks volume compared to Option C-1. This option removed an additional 321 structures from the floodplain as compared to Option C-2. 10-yr and 100-yr floodplains for this option may be viewed in Figures 6-9 and 6-10. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

6.2.3.3 Option C-3

Option C-3 includes Option C-2 and adds a 2,500 cfs pump station near the "Four Corners" area (the intersection of Boca Chica Boulevard and International Blvd.) to divert flows to Resaca de la Guerra. Option C-3 reduces the 100-year floodplain by 56 percent and the out of banks volume by 67 percent compared to the full development "no action" conditions. This corresponds to a drop in computed water surface elevations of nearly 2 feet. This is the most effective option for NMD removing an additional 830 structures from the floodplain relative to Option C-2 and 2267 structures relative to the no action scenario thus causing an enormous saving in terms of flood damages. Figures 6-11 and 6-12 show the 10-yr and 100-yr floodplains for Option C-3. Floodplains for the

2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.



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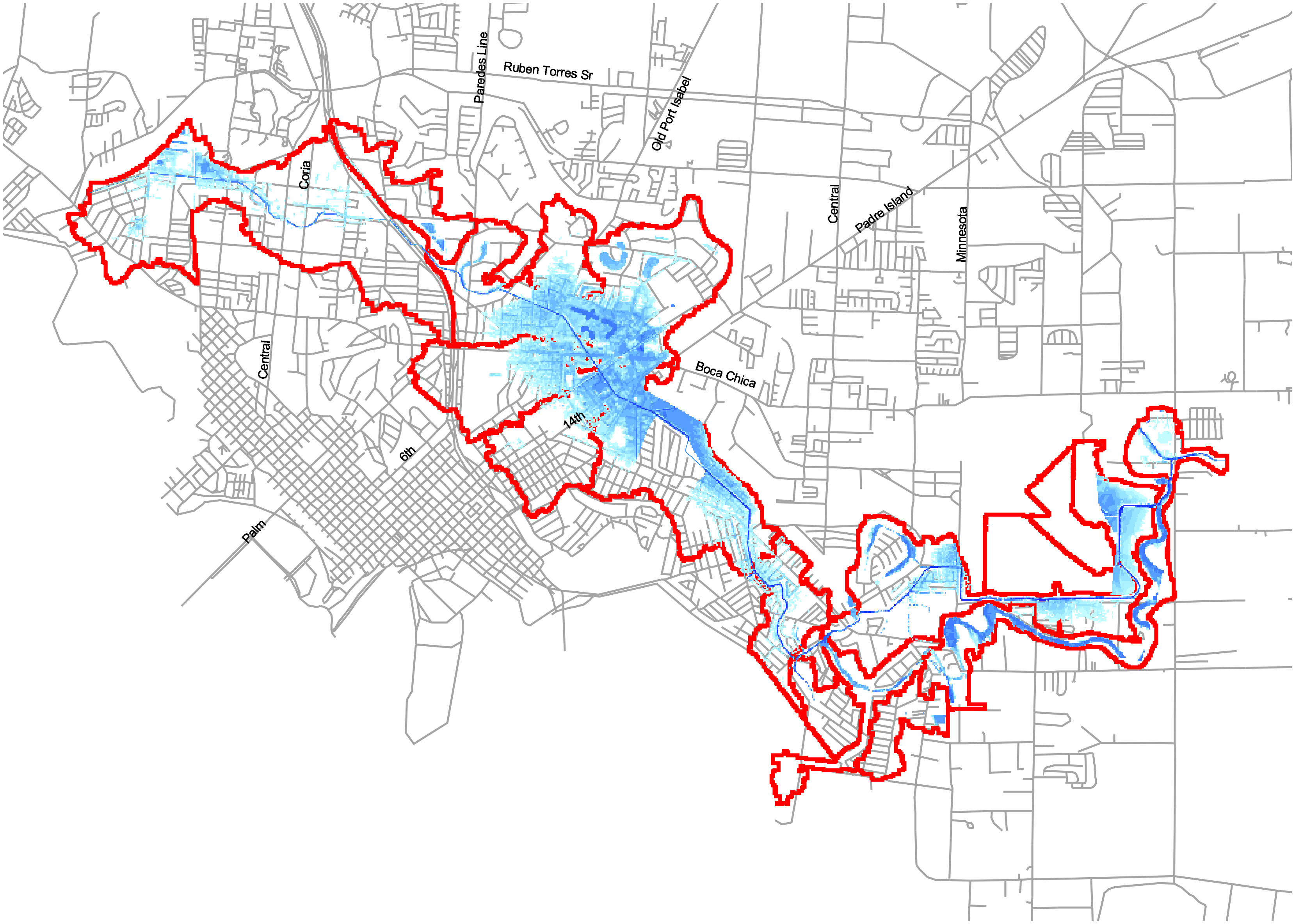
NMD Watershed
Full Development
Option NMD-C2
10-Year Floodplain Map

Figure 6-9



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

NMD
NMD Watershed
Streets

N

0 0.5 1 Miles

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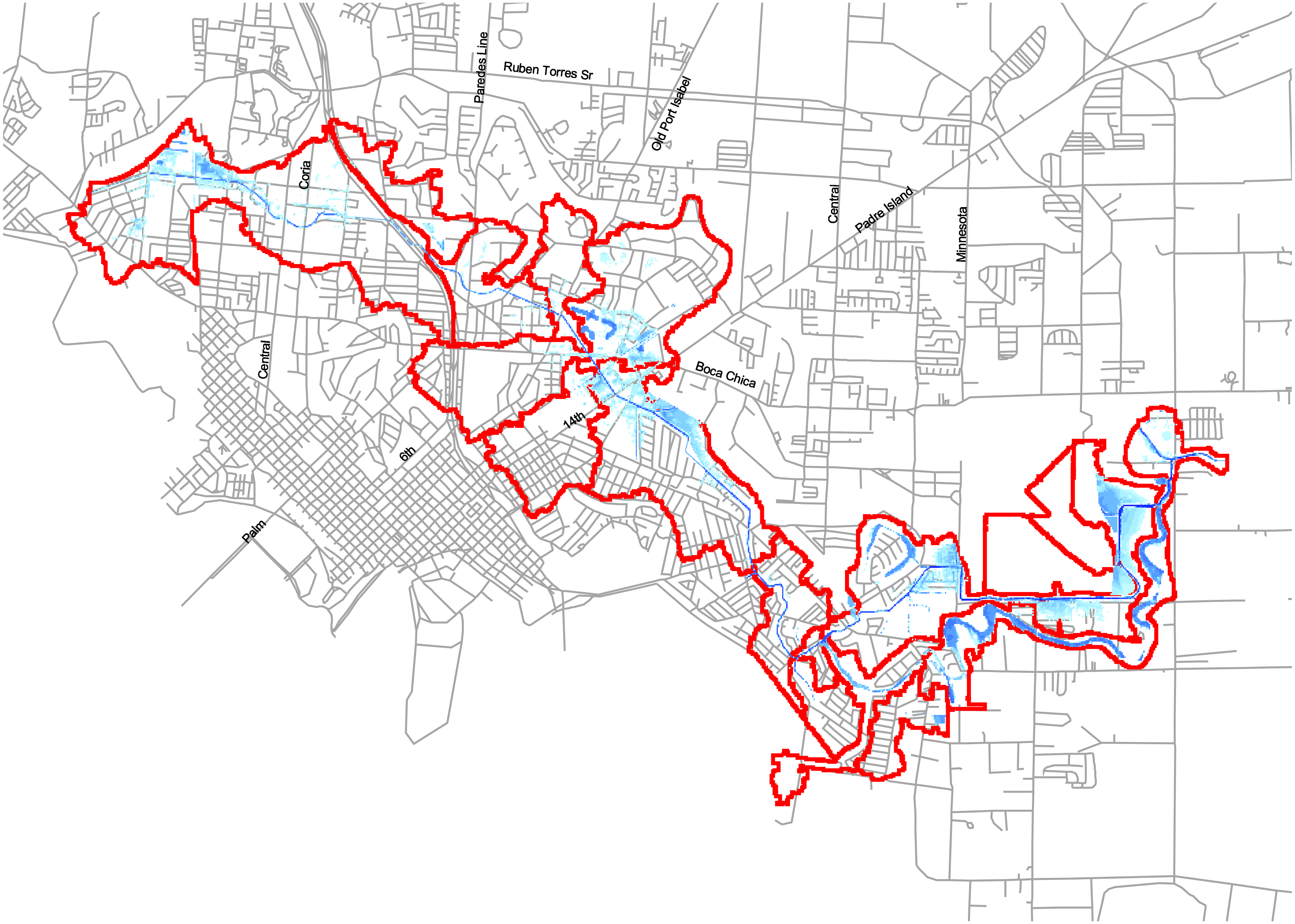
RICE

City of Brownsville, TX

NMD Watershed
Full Development
Option NMD-C2
100-Year Floodplain Map

Figure 6-10



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

NMD
NMD Watershed
Streets

N

0 0.5 1 Miles

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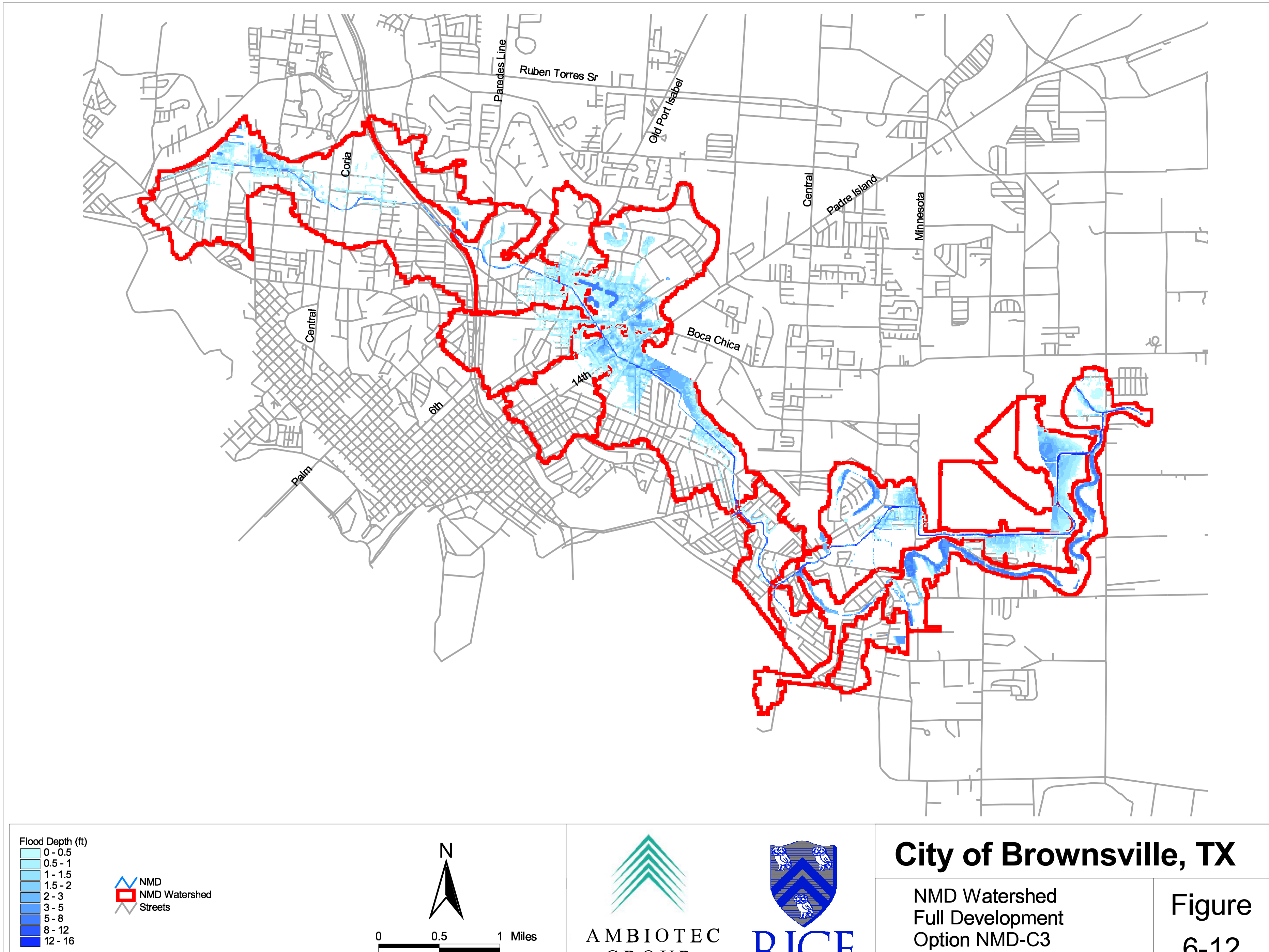
City of Brownsville, TX

NMD Watershed
Full Development
Option NMD-C3
10-Year Floodplain Map

Figure 6-11



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NMD Watershed
Full Development
Option NMD-C3
100-Year Floodplain Map

Figure 6-12

Table 6-2. North Main Drain Full Development Water Surface Elevations After the Implementation of Options A, B, C-1, C-2, and C-3

			Water Surface Elevation (ft)					
Location	**Cross Section**	**Profile**	**No Action**	**Option A**	**Option B**	**Option C-1**	**Option C-2**	**Option C-3**
Center	60720	10 year	31.94	31.94	31.99	31.98	31.92	31.98
		100 year	32.17	32.17	32.2	32.18	32.2	32.13
Mesquite Grove Sub Div	56176	10 year	31.06	30.79	30.79	30.77	30.79	30.79
		100year	31.49	31.07	31.06	31.06	31.07	31.06
Hwy 77/83	48489	10 year	29.84	29.68	29.65	29.02	25.79	24.09
		100 year	30.27	29.97	29.97	29.02	26.47	25.19
Old Port Isabel	43186	10 year	26.44	26.22	26.11	25.96	25.79	24.1
		100year	27.07	26.7	26.68	26.57	26.47	25.19
Boca Chica (DS)	40677	10 year	26.43	26.21	26.11	25.96	25.79	24.09
		100year	27.05	26.69	26.67	26.57	26.47	25.19
Btw Boca Chica and 14th	39892	10 year	26.43	26.21	26.1	25.96	25.79	24.09
		100 year	27.05	26.69	26.67	26.57	26.47	25.19
Willow	29506	10 year	24.64	24.54	24.87	24.52	24.3	23.45
		100year	26.33	25.49	26.11	25.89	25.71	24.45
Esperanza	25238	10 year	22.71	22.49	23.6	23.36	23.21	22.57
		100 year	23.99	23.13	24.46	24.33	24.23	23.48
La Posada(DS)	23862	10 year	22.5	22.21	22.03	21.92	21.85	21.76
		100year	23.63	22.69	22.52	22.45	22.39	22.33
Southmost (DS)	21363	10 year	21.81	21.35	21.66	21.58	21.52	21.48
		100 year	22.86	21.59	22.01	21.96	21.92	21.9
Minnesota Ave	15438	10 year	21.65	21.11	21.16	21.11	21.07	21.07
		100year	22.71	21.34	21.33	21.33	21.33	21.32
South Dakota	13070	10 year	21.46	20.94	21	20.94	20.9	20.9
		100 year	22.53	21.19	21.18	21.18	21.18	21.18
	6517	10 year	21.34	20.84	20.89	20.84	20.8	20.8
		100year	22.4	21.09	21.08	21.08	21.08	21.08
	3619	10 year	21.29	20.79	20.84	20.79	20.74	20.74
		100 year	22.33	21.04	21.03	21.03	21.03	21.03
Utah (US)	2738	10 year	21.18	20.69	20.75	20.69	20.65	20.65
		100 year	22.22	20.94	20.93	20.93	20.93	20.93

Table 6-3. North Main Drain Out of Bank Volume and Floodplain Area

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	**Area of Floodplain (acre)**	**Total Out of Bank Volume (ac-ft)**	**Area of Floodplain (acre)**
Full Development No Action	2630	1681	4296	2331
Option A	2266	1531	2830	1782
Option B	2221	1512	2890	1827
Option C-1	1928	1354	2502	1620
Option C-2	1589	1117	2214	1412
Option C-3	956	669	1403	1017

Table 6-4. North Main Drain Number of Flooded Structures for the Full Development 10-Year Design Storm

Water Depth (ft)	No Modifications	Option A	Option B	Option C-1	Option C-2	Option C-3
0.5 - 1.0	585	540	541	498	376	161
1.0 - 1.5	478	451	441	384	284	96
1.5 - 2.0	399	347	338	258	193	48
2.0 - 3.0	429	343	310	241	184	45
3.0 - 5.0	186	154	146	118	103	63
> 5.0	82	64	65	61	51	48
Total	2160	1900	1841	1560	1190	461

Table 6-5. North Main Drain Number of Flooded Structures for the Full Development 100-Year Design Storm

Water Depth (ft)	No Modifications	Option A	Option B	Option C-1	Option C-2	Option C-3
0.5 - 1.0	740	593	631	568	453	324
1.0 - 1.5	634	519	553	469	385	230
1.5 - 2.0	552	438	441	361	304	123
2.0 - 3.0	751	516	521	402	361	108
3.0 - 5.0	396	242	244	201	183	79
> 5.0	118	74	77	74	68	59
Total	3191	2382	2467	2075	1754	924

6.3 Cameron County Drainage District No. 1 Ditch

The Cameron County Drainage District No. 1 Ditch No. 1 watershed is heavily inundated during large rainstorm events. During a 100-year event, the floodplain will cover approximately 3,650 acres at current development conditions and 4,338 acres for the full development model (Table 4-16). As mentioned in section 4.0, it is not the extent of the floodplain that is drastically increased, but the water surface elevations that are severely impacted.

Currently, the majority of the flooded area is undeveloped agricultural or farmland regions. However, the expected growth that is projected to occur in this region prevents this from becoming a viable option. A much more feasible, and cost effective, non-structural option for CCDD1 is to impose runoff controls that limit the runoff from future developments to be less than or equal to the runoff under existing undeveloped conditions and prohibit the raising of existing water surface elevations from new development. This reduces the full development floodplains to the existing floodplains and reduces significantly the costs of structural options needed to alleviate flooding in the watershed.

Over twenty different structural alternatives were considered for CCDD1 including detention ponds, channel modifications and hydraulic structure improvements. Similar to NMD, these alternatives were grouped in a series of options (A, B, and C) to facilitate the estimation of costs and benefits. Once again each subsequent option builds on the previous set of alternatives by including an additional set of alternatives. The three specific alternatives include:

- Option A: Runoff controls on future development; 12 detention ponds ranging in storage capacity from 90 ac-ft near the intersection of the UP railroad track, to 720 ac-ft at FM 802; the installation of two side weirs; and the replacement of the weir structure at Paredes Line Road

- Option B: Option A plus channel improvements from Parades Line Road to FM 802

- Option C: Option B plus channel improvements from FM 802 to outfall and the replacement of 6 culverts

6.3.1 Option A

Option A includes runoff controls on future developments, the construction of 14 detention ponds that would store diverted water from the drainage ditch, and the improvement of the hydraulic structure at Paredes Line Road to increase the storage of the ditch upstream. Detentions ponds are an economical alternative due to the amount of land available within this watershed. The detention ponds proposed for the CCDD1 watershed would also be offline detention ponds with a hydraulic structure, such as a weir, governing the amount of flow entering the pond. The detention ponds for the CCDD1 watershed are listed in Table 6-6 and shown in Figure 6-13. In addition to the construction of the 14 detention ponds, a culvert improvement is also proposed at Paredes Line Road. Currently, there is a 4' x 4' drop structure at this crossing with an elevation of 16 feet and a flow line of 8.2 feet. This current drop structure allows for the ponding of water within the ditch, reducing the effective storage capacity of the channel. This drop structure was removed in option A and replaced with an open span bridge.

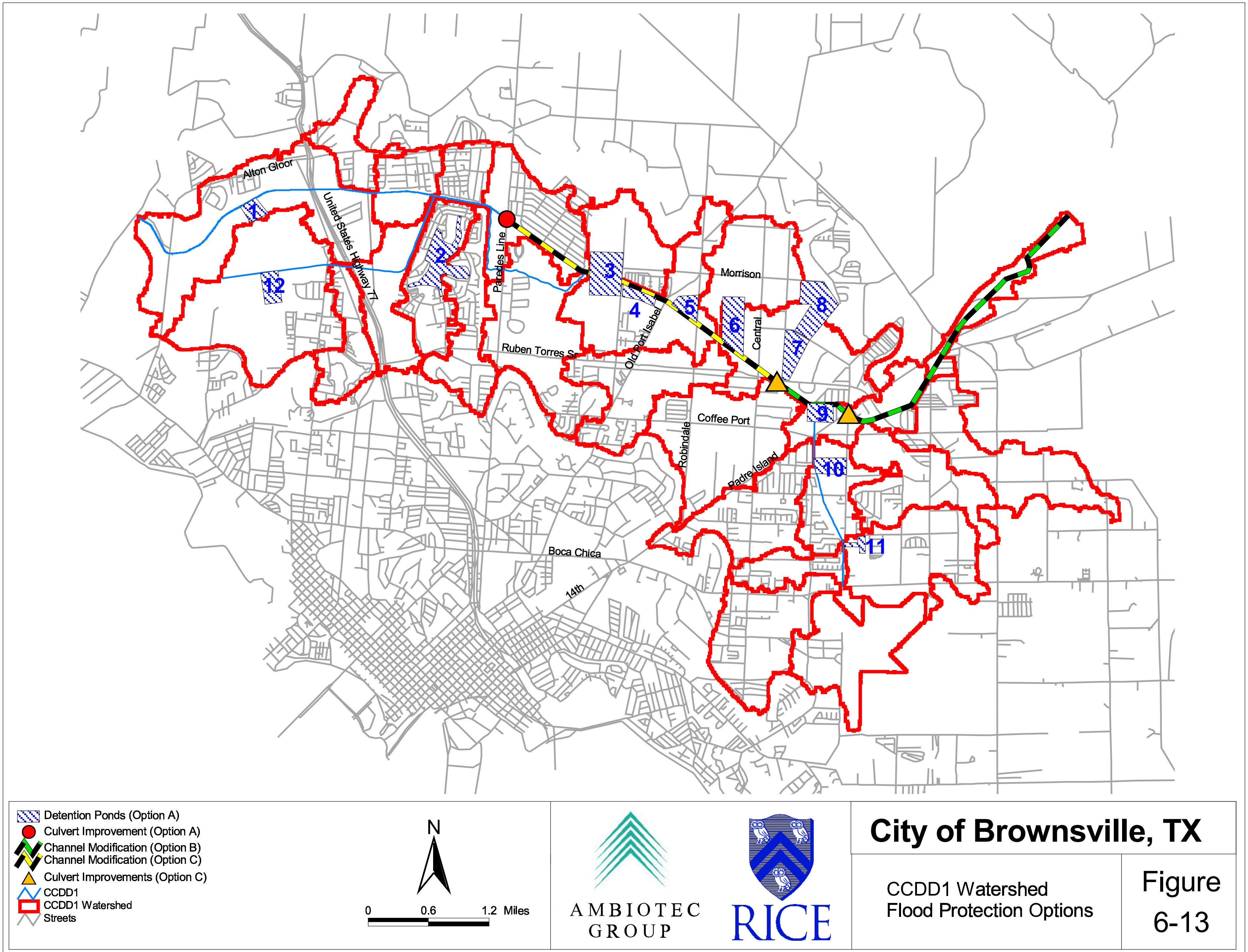
The implementation of Option A in the models reduces the overland coverage of the 100-year floodplain by approximately 41 percent (10-yr, 69%) and the out of bank volume by approximately 4 percent (10-yr, 6%) relative to the existing development

Table 6-6. Proposed Detention Ponds for CCDD1

Map Point	Location	Acreage (acres)	Capacity (ac-ft)
1	Upstream of Union Pacific Railroad	50	240
2	Brownsville Golf Center	60	180
3	(2) - Dana Rd & CCDD1 crossing	65	650
4	Towne North	50	500
5	CCDD1 Main Office	25	300
6	Near FM 802 and Robindale Rd	53	500
7	FM 802 and CCDD1 crossing	100	720
8	FM 802 and CCDD1 crossing	100	720
9	(2) - Coffee Port Rd and CCDD1 (Public Works Yard)	10	100
10	Minnesota Ave just north of Austin Rd	29	200
11	Minnesota Ave just north of Boca Chica Blvd	26	260
12	Nopalitos Drain - Upstream of Union Pacific Railroad	50	240

conditions. 10-year and 100-year floodplains for this option may be viewed in Figures 6-14 and 6-15. For the full development condition the overland coverage of the 100-year floodplain is reduced by 50 percent (10-yr, 75%) and the out of bank volume is reduced by 29 percent (10-yr, 34%). This reduction in floodplain area removed 4809 structures from the 100-yr floodplain relative to the no action full development conditions. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

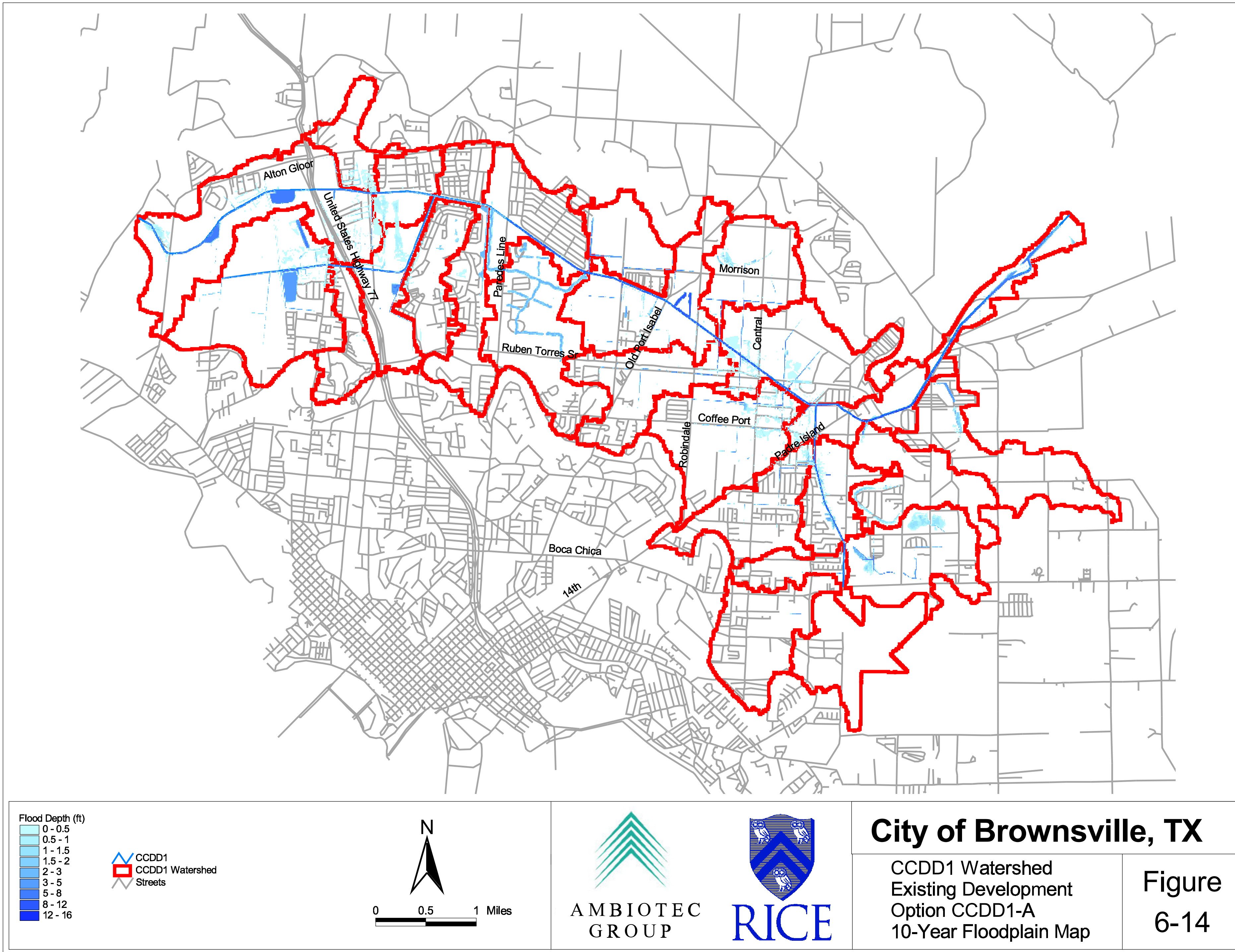
A major contributing factor to the reduction in the floodplain area is the effect of the runoff controls that limit future flows to existing values. The effects of implementing Option A without runoff controls may be viewed in Tables 6-11 and 6-12 and Figures 6-16 and 6-17 (Appendix C). This table reveals that implementing runoff controls on future developments within the watershed reduce the total number of flooded structures by 35%.



City of Brownsville, TX

CCDD1 Watershed
Flood Protection Options

Figure 6-13



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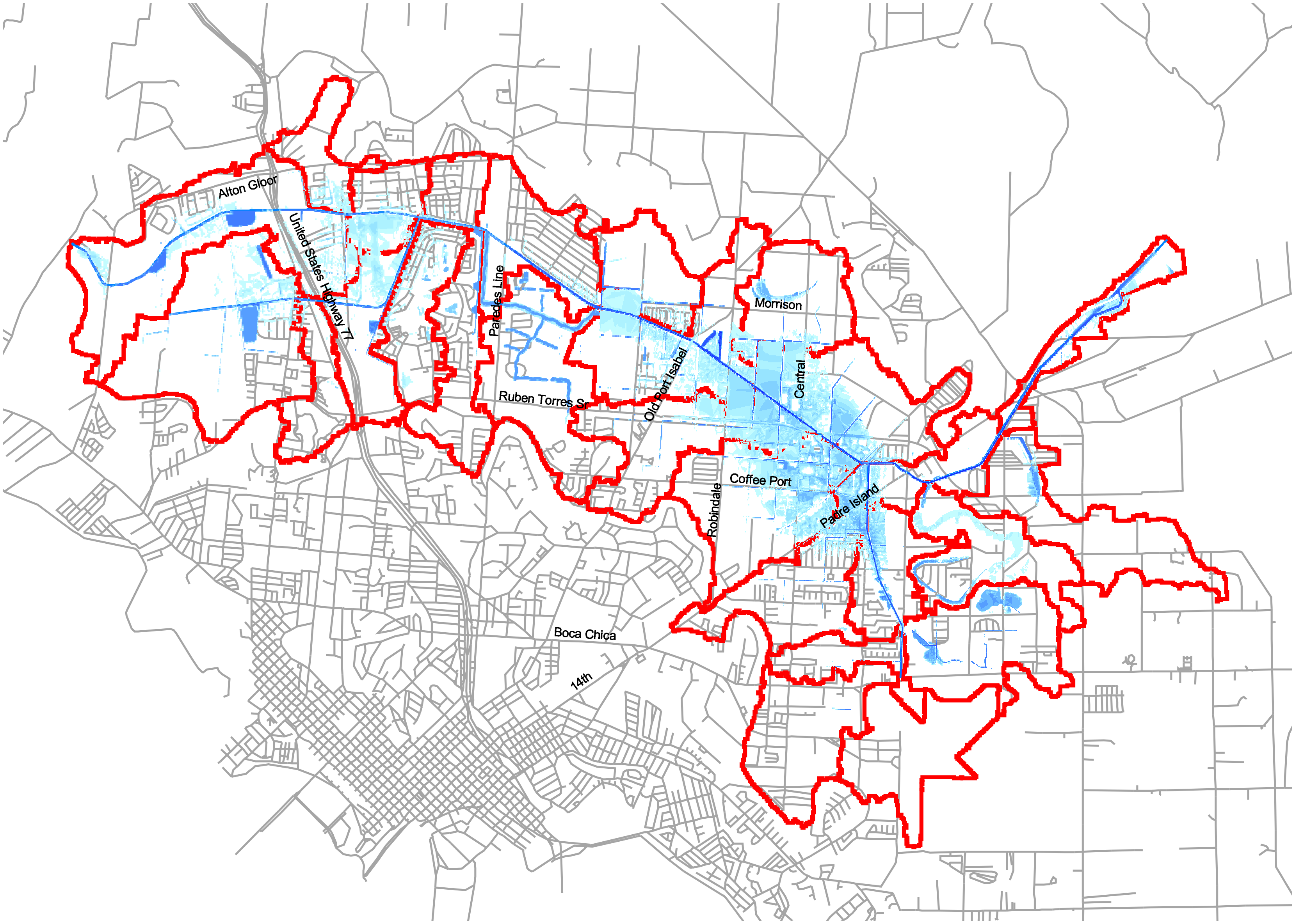
CCDD1 Watershed
Existing Development
Option CCDD1-A
10-Year Floodplain Map

Figure 6-14



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

CCDD1
CCDD1 Watershed
Streets

N

0 0.5 1 Miles

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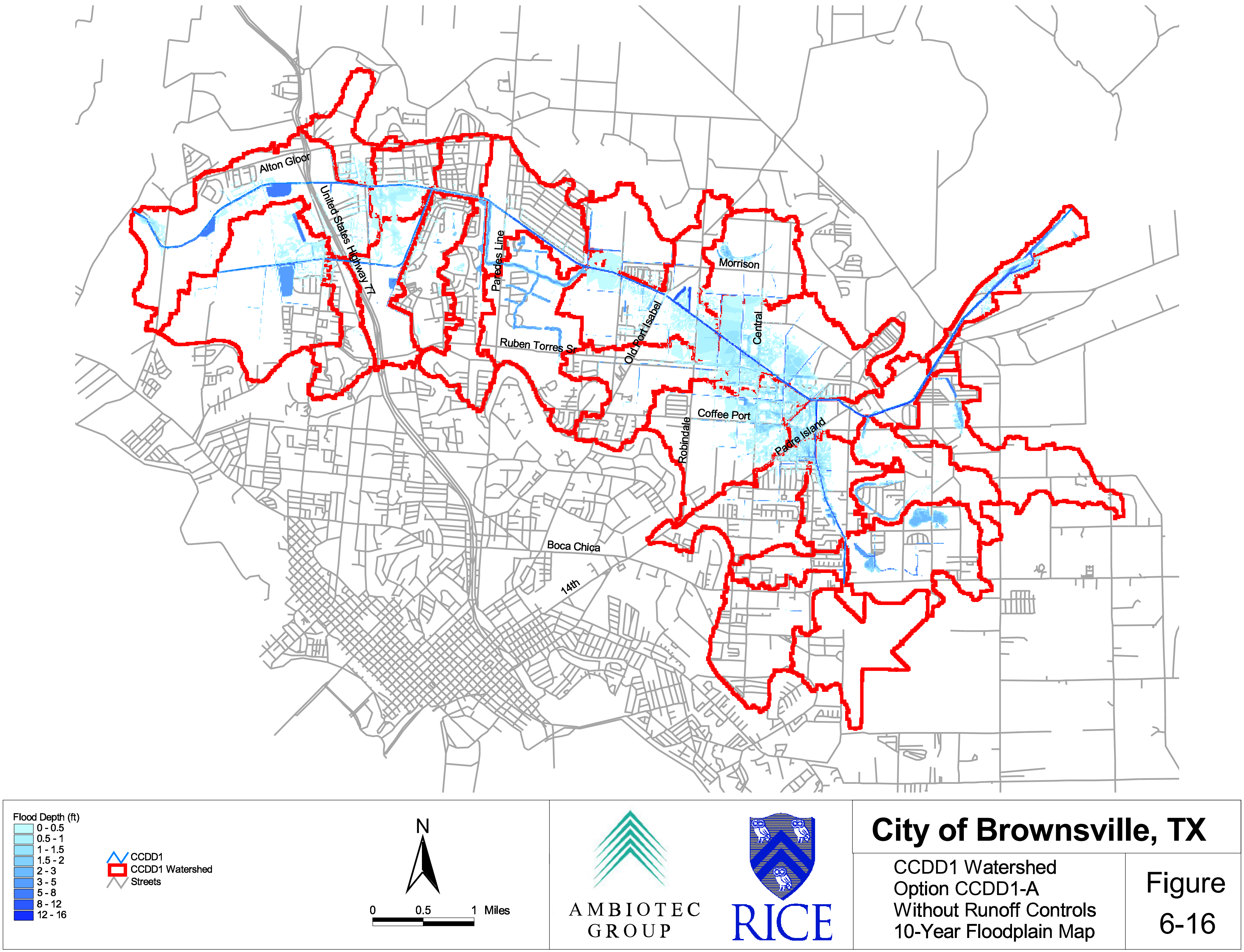
City of Brownsville, TX

CCDD1 Watershed
Existing Development
Option CCDD1-A
100-Year Floodplain Map

Figure 6-15



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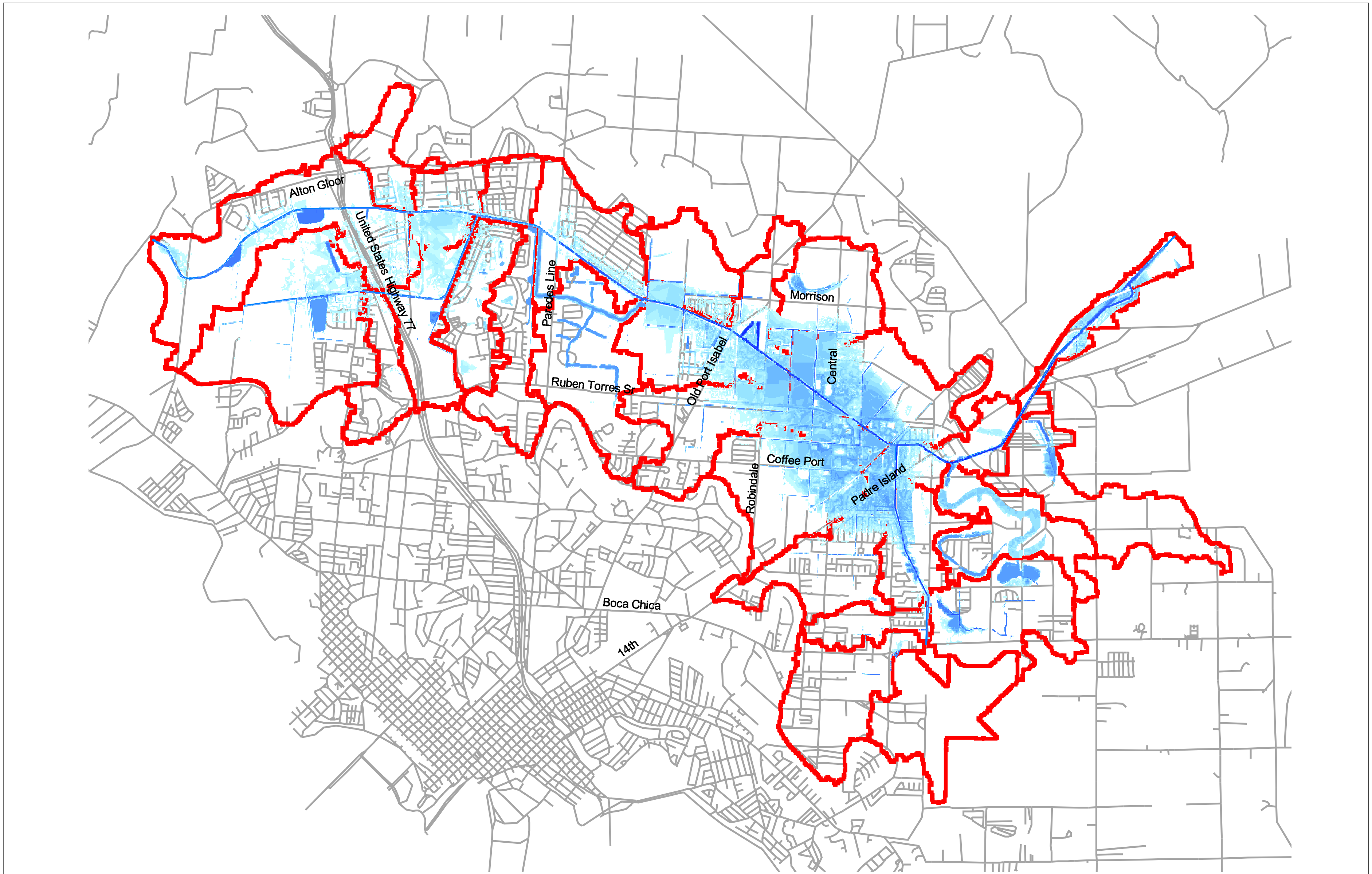
CCDD1 Watershed
Option CCDD1-A
Without Runoff Controls
10-Year Floodplain Map

Figure 6-16



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

CCDD1
CCDD1 Watershed
Streets

N

0 0.5 1 Miles

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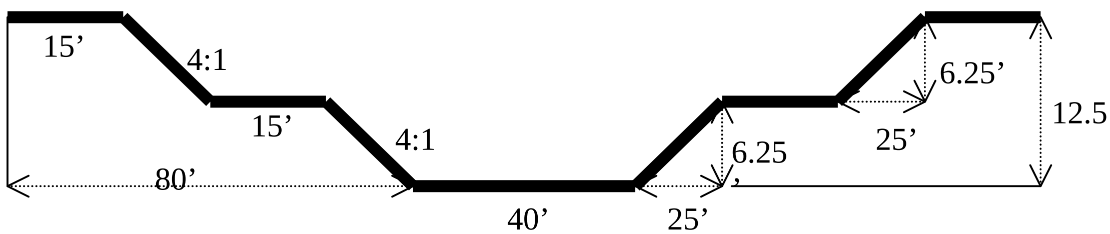
CCDD1 Watershed
Option CCDD1-A
Without Runoff Controls
100-Year Floodplain Map

Figure 6-17

6.3.2 Option B

Option B includes all changes completed in Option A in addition to channel improvements to the CCDD1 ditch from Paredes Line Road to FM 802 as seen in Figure 6-13. The channel improvement includes widening the channel assuming a right-of-way of 200 feet, and concrete lining the channel to reduce the energy losses. The specifications of the channel modification are shown in Figure 6-18.

Figure 6-18. CCDD1 Channel Modification Specifications



The purpose of the double trapezoidal channel is to facilitate maintenance provide a recreational walk/run/bike path during low flow conditions. The features described in Option B result in a reduction of the existing development 100-year floodplain by 47 percent (10-yr, 74%) and the full development floodplain by 56 percent (10-yr, 79%). The out of banks flooding volume is reduced by 65% and 74% (10-yr, 71% and 80%) respectively for the existing and full development scenarios. In this option 5040 structures were removed from the 100-yr floodplain relative to the full development no action scenario. This represents an additional reduction of 231 structures from the floodplain as compared to Option A. The 10-yr and 100-yr floodplains for this option may be viewed in Figures 6-19 and 6-20. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

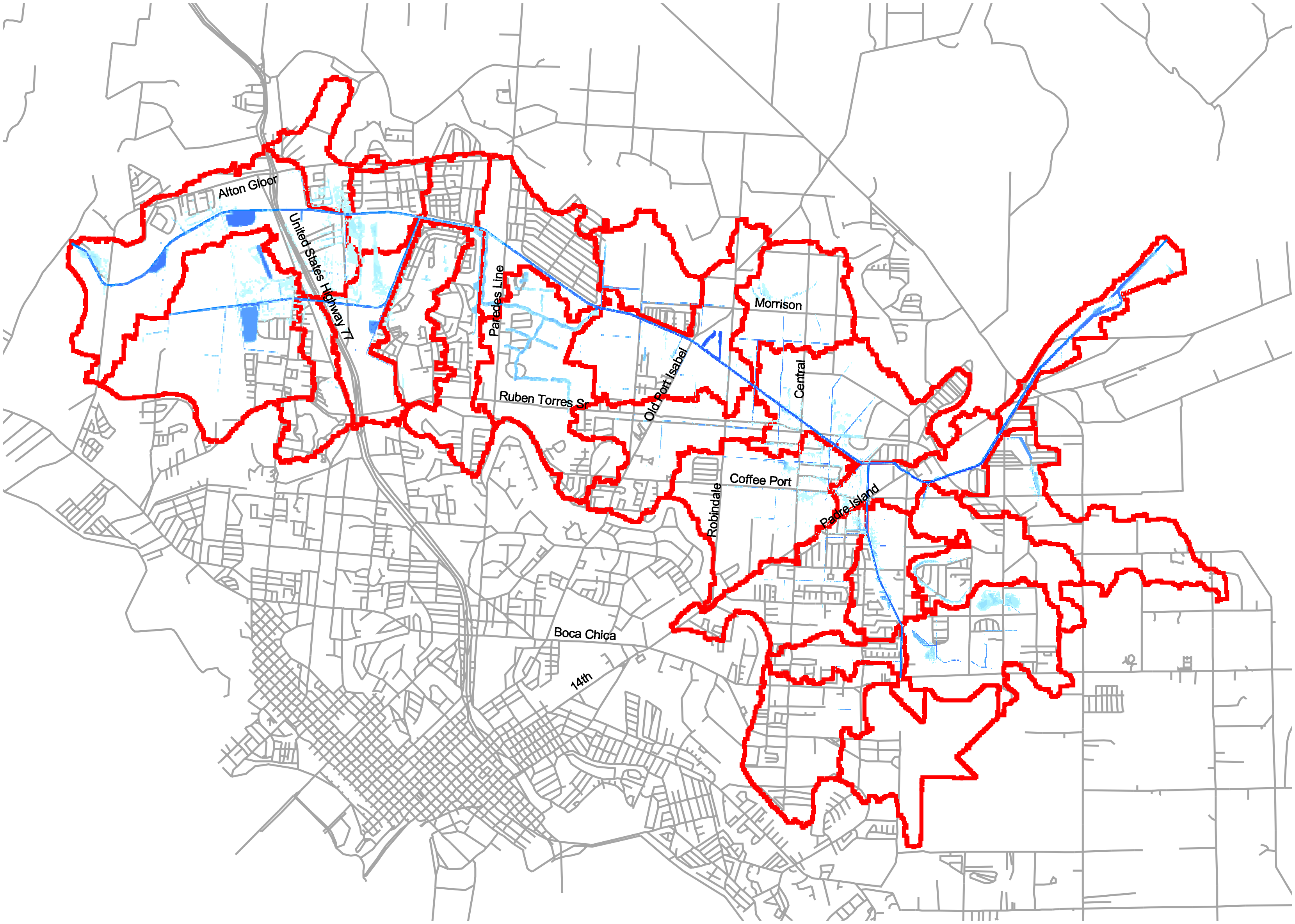
The effects of implementing the structural components of Option B without runoff controls may be viewed in the 10-year and 100-year floodplains (Figures 6-21 and 6-22). This alternative provides flood protection for 1486 fewer homes than the same option implemented with runoff controls. Thus runoff controls account for an additional 39% reduction in the number of flooded structures for this option.

6.3.3 Option C

Option C includes all features described in Option A and Option B with an extension of the channel modification down to the outlet. In addition to this, two bridges in the downstream portion of the drainage ditch will be elevated so that they do not create an obstruction to flow during large storm events. The Option B channel modification was effective in conveying water through Robindale Road; but the increase in the roughness and decrease of channel volume at FM 802, where the channel modification ended, created a back up in the water surface elevation that could not be handled by the proposed detention ponds. Therefore, it is necessary to modify the channel to the outlet of the drainage system. It is also important to extend the channel modification past the fluvial deposits of Resaca del Rancho Viejo, along Minnesota Avenue, to prevent a pooling effect within the “bowl” of the CCDD1 watershed, which is responsible for extensive flooding. The geometry of the channel modification extension resembles that of Option B as seen in Figure 6-4, and is lined.

The second modification in Option C involves the elevation of two bridge structures in the downstream portion of the ditch. Currently, there are two road crossings at FM 802 and International Blvd. that are creating large head losses due to road decks that are too low and thus cause a backup of flow during large storm events. In order to make these crossings hydraulically efficient in conjunction with the channel modification, the bridges at FM 802 and International Blvd. need to be elevated. While other hydraulic structures along the channel will be recommended for improvements, the mentioned bridges are the leading cause of flooding in this region due to the low elevation of the deck.

The modeling runs for Option C show a reduction in the existing development 100-year floodplain by 79 percent (10-yr, 82%) and the full development floodplain by 82 percent (10-yr, 86%). The existing and full development out of banks volumes for the 100-year event are reduced by 83 and 88 percent, respectively (10-yr, 78%, 85%). The number of houses taken out of the 100-year floodplain has increased to 6650 (10-yr, 4254) relative to the no action scenario and 1610 (10-yr, 212) relative to Option B. The 10-yr and 100-yr floodplains for this run may be viewed in Figures 6-23, 6-24, 6-25, and 6-26. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

CCDD1
CCDD1 Watershed
Streets

N

0 0.5 1 Miles

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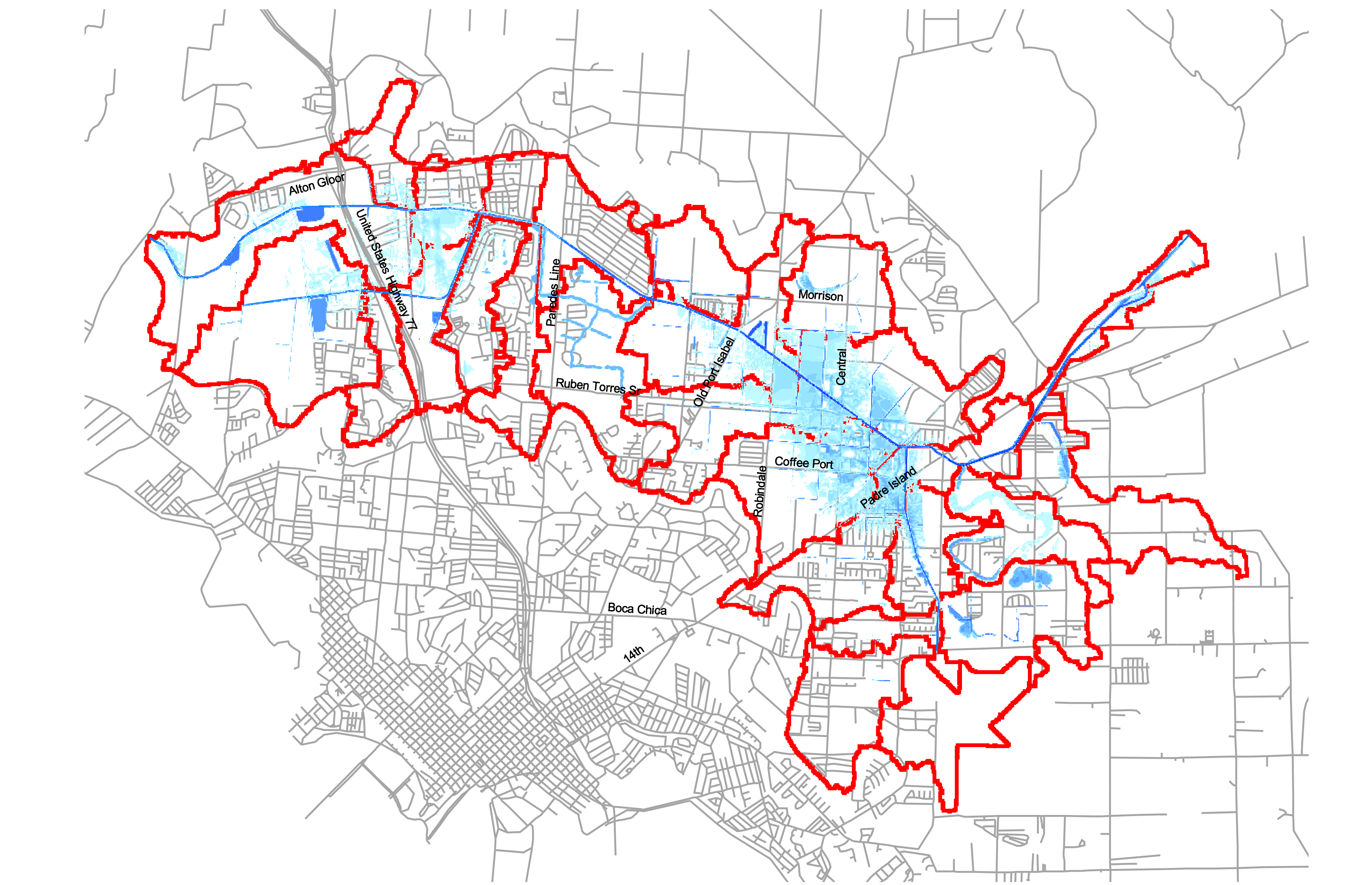
CCDD1 Watershed
Existing Development
Option CCDD1-B
10-Year Floodplain Map

Figure 6-19



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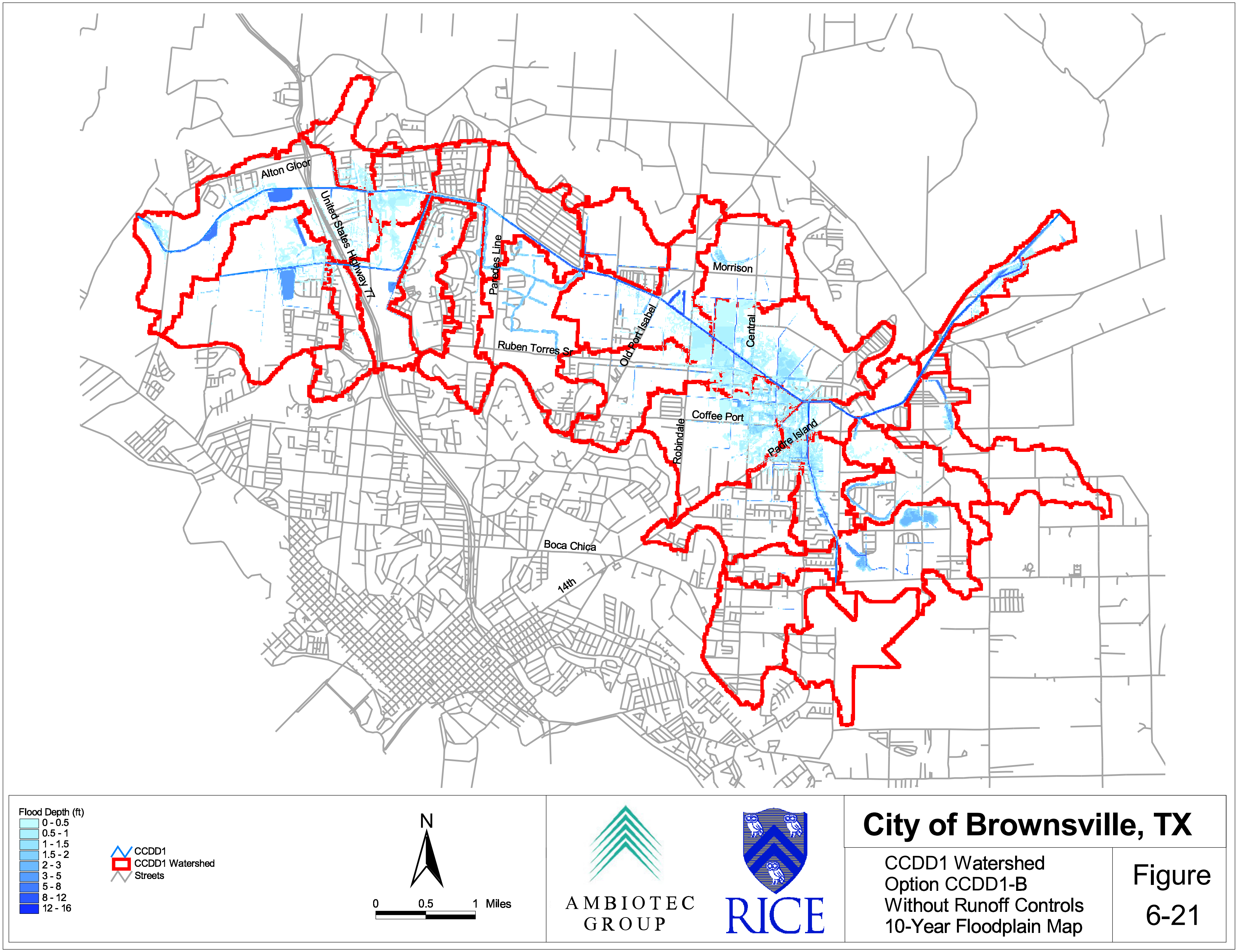
CCDD1 Watershed
Existing Development
Option CCDD1-B
100-Year Floodplain Map

Figure 6-20



Alton Gloor
United States Highway 77
Paredes Line
Ruben Torres Sr
Old Port Isabel
Morrison
Central
Coffee Port
Robindale
Padre Island
Boca Chica
14th
Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16
CCDD1
CCDD1 Watershed
Streets
N
0
0.5
1
Miles
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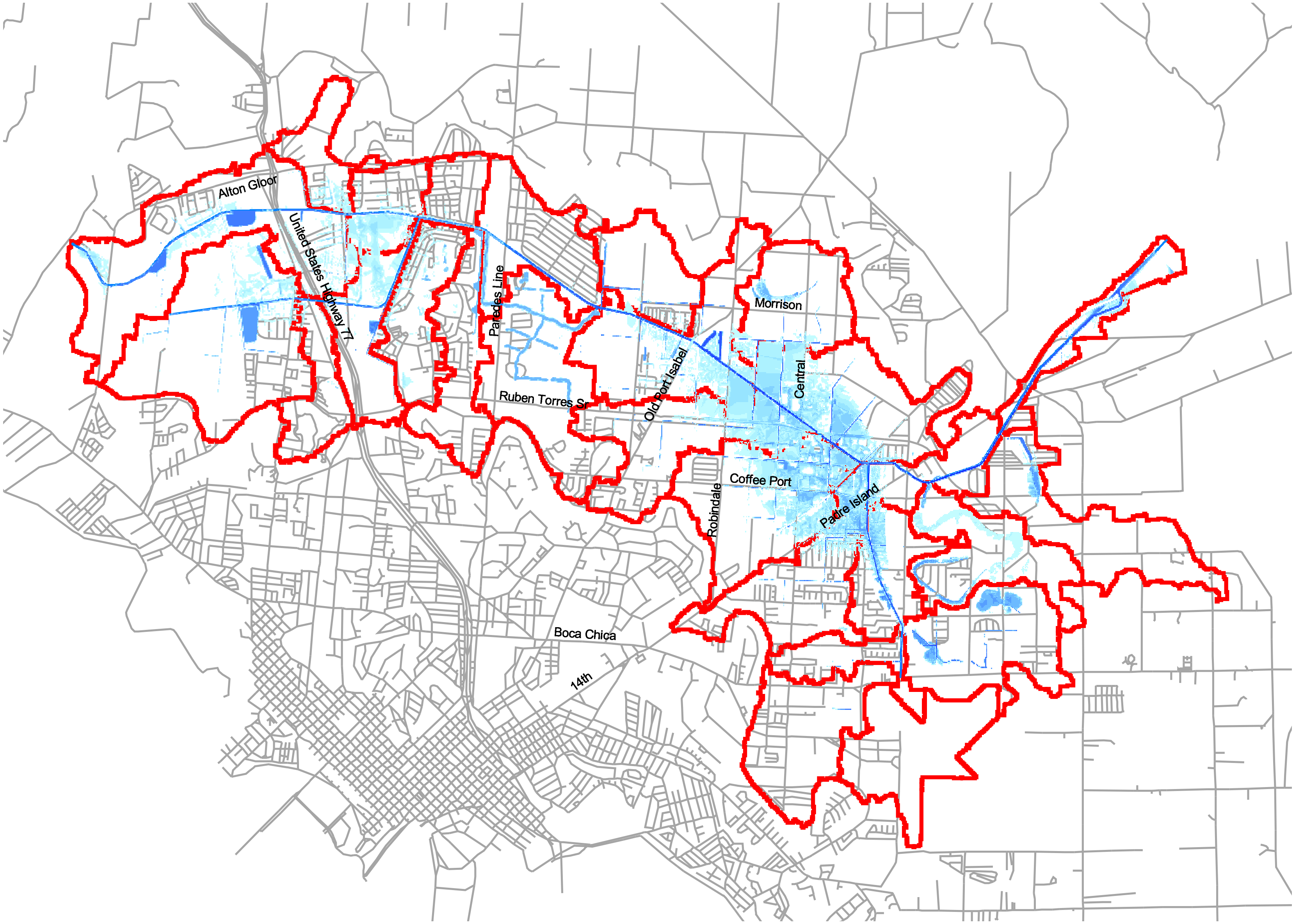
City of Brownsville, TX
CCDD1 Watershed
Option CCDD1-B
Without Runoff Controls
10-Year Floodplain Map
Figure
6-21



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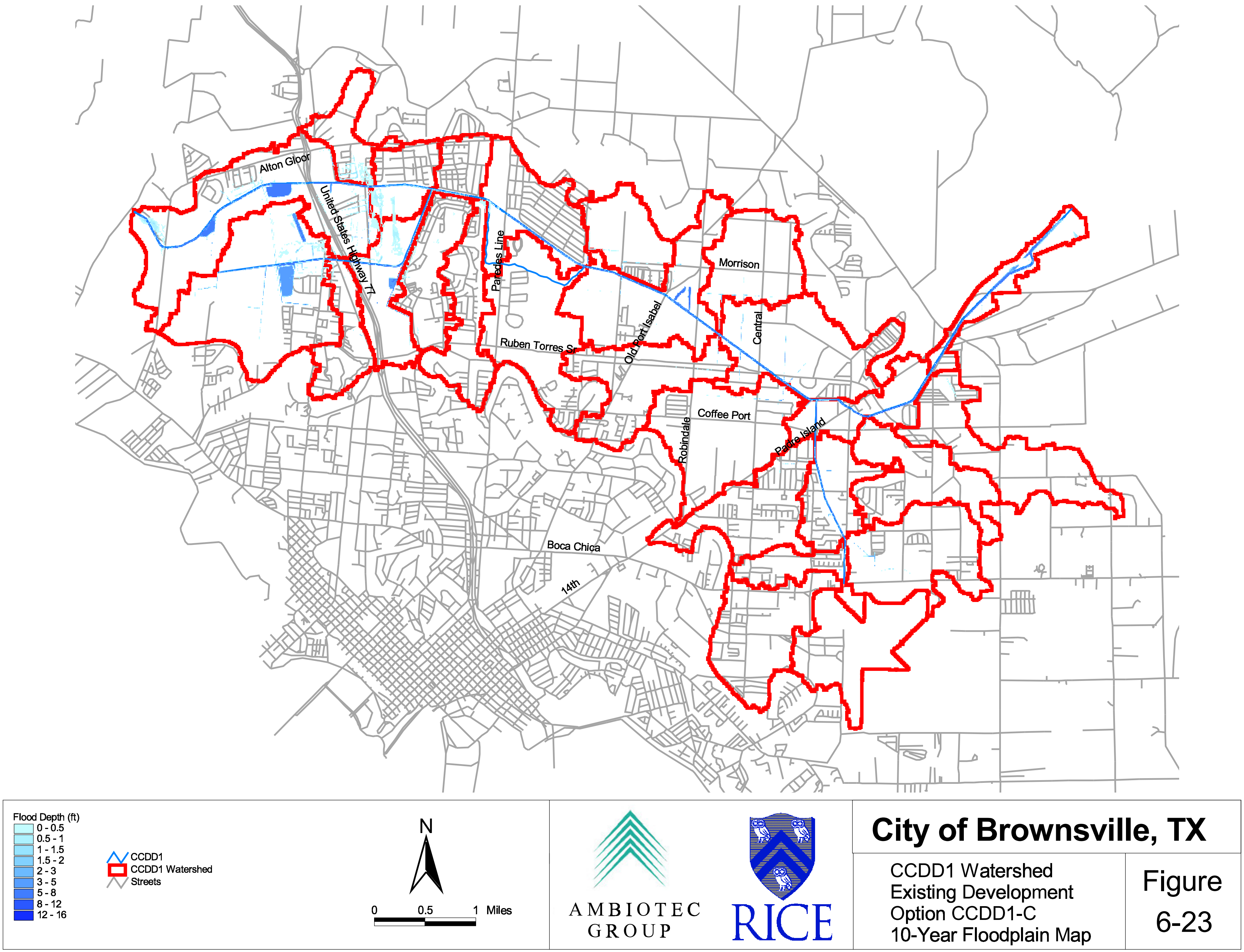
CCDD1 Watershed
Option CCDD1-B
Without Runoff Controls
100-Year Floodplain Map

Figure 6-22



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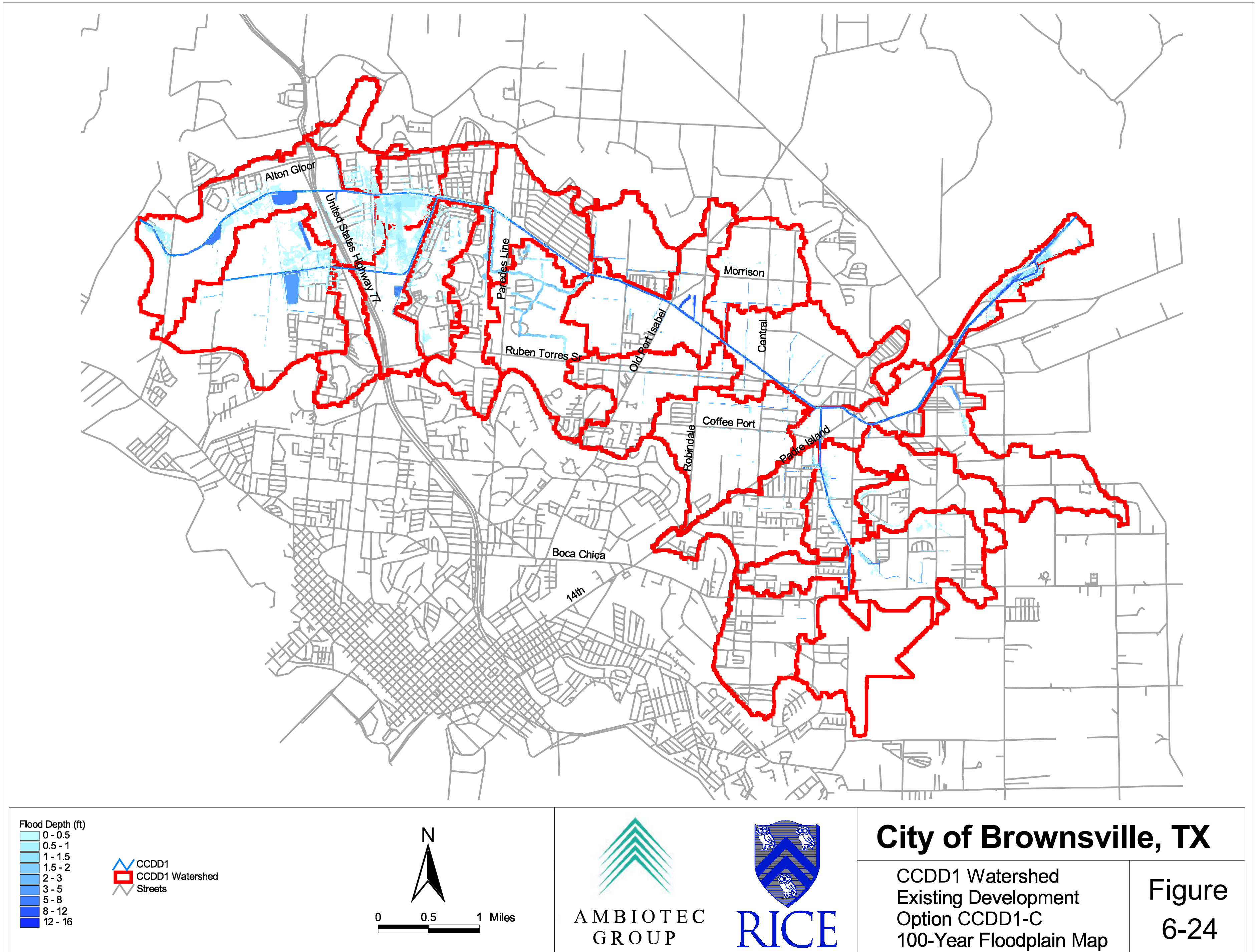
CCDD1 Watershed
Existing Development
Option CCDD1-C
10-Year Floodplain Map

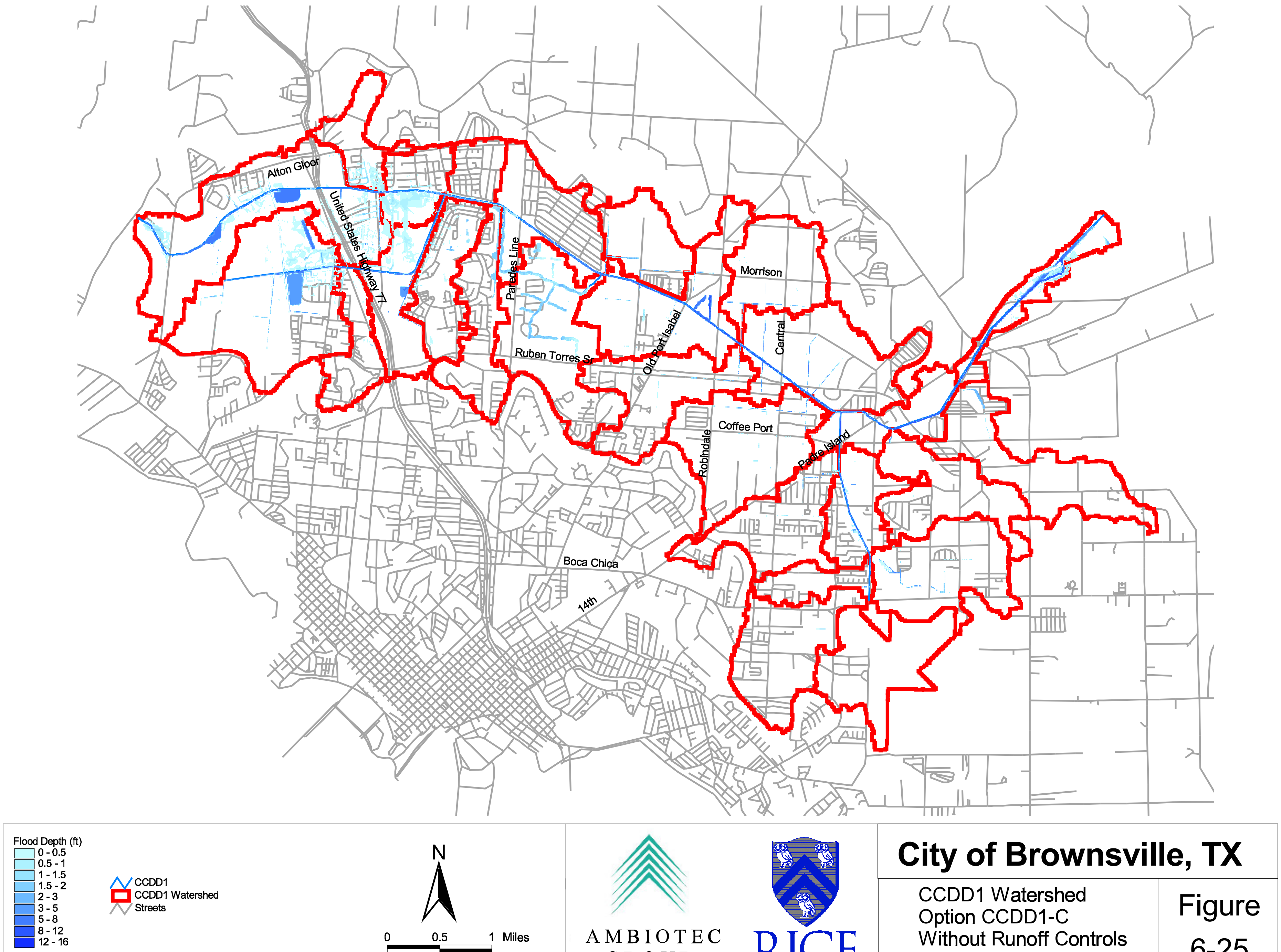
Figure 6-23



City of Brownsville, TX

CCDD1 Watershed
Existing Development
Option CCDD1-C
100-Year Floodplain Map

Figure
6-24



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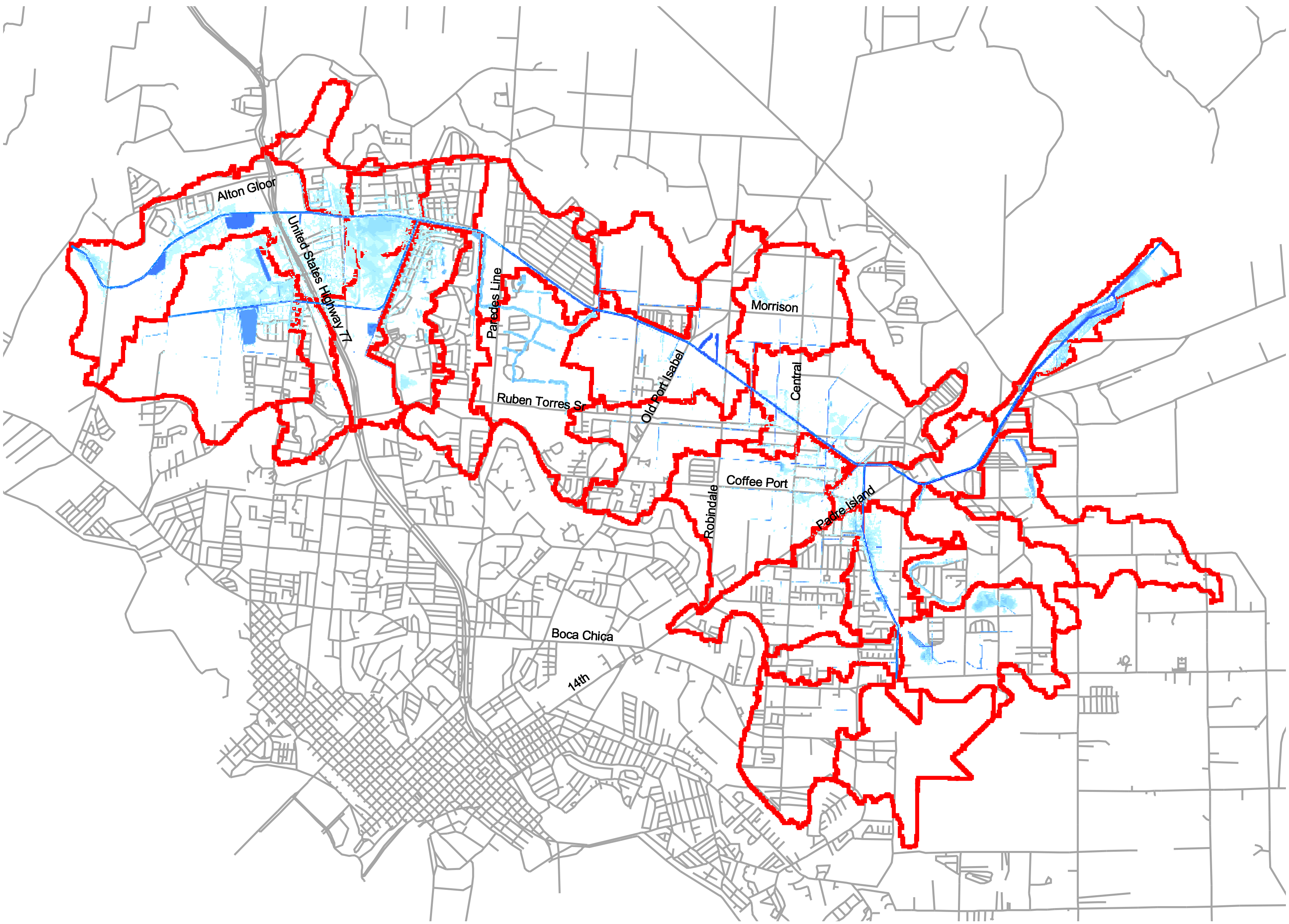
CCDD1 Watershed
Option CCDD1-C
Without Runoff Controls
10-Year Floodplain Map

Figure 6-25



Flood Depth (ft)
- 0 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2
- 2 - 3
- 3 - 5
- 5 - 8
- 8 - 12
- 12 - 16

- CCDD1
- CCDD1 Watershed
- Streets

N

0 0.5 1 Miles

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CCDD1 Watershed
Option CCDD1-C
Without Runoff Controls
100-Year Floodplain Map

Figure 6-26

Table 6-7. CCDD1 Water Surface Elevations after Implementation of Options A, B, & C

Location	Cross Section	Profile	Water Surface Elevation (ft)				
			Existing No Action	Full No Action	Option A	Option B	Option C
Beginning	59386.47	10 year	27.24	27.32	27.22	27.06	27.06
		100 year	27.35	27.47	27.35	27.26	27.26
Alton Gloor	57023.95	10 year	22.94	23.27	22.94	22.66	22.66
		100 year	23.45	23.76	23.45	23.11	23.11
HW 77	48213.56	10 year	21.39	21.77	20.96	20.76	20.76
		100 year	21.86	22.12	21.37	20.97	21
Pablo Kisel	44494.17	10 year	20.94	20.92	19.74	18.65	18.59
		100 year	21.42	21.29	20.8	20.25	20.27
Paredes Line	37764.99	10 year	17.77	18.24	17.88	15.3	14.45
		100 year	18.72	19.28	18.85	16.76	15.49
Dana Ave	30766.94	10 year	17.32	17.81	17.32	14.67	12.27
		100 year	18.41	19.02	18.41	16.45	14.15
Old Port Isabel	28290.97	10 year	16.98	17.54	16.87	14.55	11.4
		100 year	18.2	18.9	18.19	16.34	13.58
Robindale	26203.98	10 year	16.9	17.51	16.9	14.52	11.21
		100 year	18.17	18.87	18.17	16.3	13.43
Central Ave	22326.12	10 year	16.8	17.45	16.8	14.47	10.86
		100 year	18.13	18.85	18.13	16.27	13.19
FM 802	21389.29	10 year	16.78	17.44	16.78	14.43	10.79
		100 year	18.11	18.84	18.11	16.27	13.15
Padre Is. & Minnesota	16499.11	10 year	16.35	17.23	16.35	13.87	10.21
		100 year	17.99	18.78	17.99	15.79	12.66
FM 802	14785.25	10 year	16	16.99	16	13.48	9.94
		100 year	17.77	18.66	17.77	15.46	12.43
Capt. Donald L Faust	10256.28	10 year	11.36	13.51	11.36	9.17	9.13
		100 year	12.89	14.77	12.89	10.9	11.79
FM 511	9272.439	10 year	10.94	12.38	10.94	8.53	8.97
		100 year	12.51	13.47	12.51	10.46	11.67
Port Isabel Hwy	6391.508	10 year	9.12	10.62	9.12	7.22	7.43
		100 year	10.22	11.19	10.22	8.77	9.39

Table 6-8. CCDD1 Out of Bank Flooding Volume and Floodplain Area After Implementation of Options A, B, & C

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	3227	2462	6459	3650
Future Development	4634	3053	8793	4338
Option A	3047	759	6208	2165
Option B	942	635	2286	1919
Option C	690	441	1071	765

Table 6-9. CCDD1 Number of Flooded Structures for the 10-yr Event After Implementation of Option A, B, & C

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A	Option B	Option C
0.5 - 1.0	207	1179	172	156	111
1.0 - 1.5	103	901	73	71	41
1.5 - 2.0	84	861	59	46	19
2.0 - 3.0	30	995	69	66	34
3.0 - 5.0	8	368	111	100	66
> 5	0	296	129	118	74
Total	432	4599	613	557	345

Table 6-10. CCDD1 Number of Flooded Structures for the 100-yr Event After Implementation of Option A, B, & C

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A	Option B	Option C
0.5 - 1.0	511	1492	1064	1001	279
1.0 - 1.5	344	1179	654	558	113
1.5 - 2.0	290	913	266	229	56
2.0 - 3.0	277	1553	204	203	59
3.0 - 5.0	83	1774	172	150	93
> 5	2	451	195	182	113
Total	1507	7363	2554	2323	713

Table 6-11. CCDD1 Water Surface Elevations after Implementation of Options A, B, and C Without Runoff Controls

			Water Surface Elevation (ft)				
Location	**Cross Section**	**Profile**	**Existing No Action**	**Full No Action**	**Option A**	**Option B**	**Option C**
Beginning	59386.47	10 year	27.24	27.32	27.21	27.19	27.22
		100 year	27.35	27.47	27.36	27.36	27.36
Alton Gloor	57023.95	10 year	22.94	23.27	22.94	22.94	22.94
		100 year	23.45	23.76	23.4	23.4	23.4
HW 77	48213.56	10 year	21.39	21.77	20.97	20.99	21.02
		100 year	21.86	22.12	21.29	21.31	21.3
Pablo Kisel	44494.17	10 year	20.94	20.92	19.97	19.94	19.96
		100 year	21.42	21.29	20.68	20.71	20.71
Paredes Line	37764.99	10 year	17.77	18.24	18.18	16.23	14.79
		100 year	18.72	19.28	19.39	17.57	16.24
Dana Ave	30766.94	10 year	17.32	17.81	16.59	15.9	13.39
		100 year	18.41	19.02	17.59	17.24	15.36
Old Port Isabel	28290.97	10 year	16.98	17.54	16.08	15.76	12.67
		100 year	18.2	18.9	17.23	17.15	14.92
Robindale	26203.98	10 year	16.9	17.51	15.96	15.73	12.49
		100 year	18.17	18.87	17.21	17.14	14.79
Central Ave	22326.12	10 year	16.8	17.45	15.75	15.69	12.21
		100 year	18.13	18.85	17.14	17.12	14.6
FM 802	21389.29	10 year	16.78	17.44	15.72	15.68	12.16
		100 year	18.11	18.84	17.13	17.12	14.57
Padre Is. & Minnesota	16499.11	10 year	16.35	17.23	15.34	15.32	11.69
		100 year	17.99	18.78	16.96	16.93	14.22
FM 802	14785.25	10 year	16	16.99	15	14.98	11.43
		100 year	17.77	18.66	16.73	16.7	13.98
Capt. Donald L Faust	10256.28	10 year	11.36	13.51	11.75	11.73	10.78
		100 year	12.89	14.77	13.34	13.32	13.39
FM 511	9272.439	10 year	10.94	12.38	10.73	10.71	10.66
		100 year	12.51	13.47	12.23	12.21	13.28
Port Isabel Hwy	6391.508	10 year	9.12	10.62	9.34	9.32	8.9
		100 year	10.22	11.19	10.52	10.51	10.65

Table 6-12. CCDD1 Out of Bank Flooding Volume and Floodplain Area After Implementation of Option A, B, & C Without Runoff Controls

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	3227	2462	6459	3650
Future Development	4634	3053	8793	4338
Option A	1755	1594	4052	2864
Option B	1568	1405	3873	2756
Option C	848	613	1534	1165

Table 6-13. CCDD1 Number of Flooded Structures After Implementation of Options A, B, & C With and Without Runoff Controls During a 100-Year Event

Water Depth (ft)	Option A	Option A w/o Runoff Controls	% Change	Option B	Option B w/o Runoff Controls	% Change	Option C	Option C w/o Runoff Controls	% Change
0.5 - 1.0	1064	1082	2%	1001	1049	5%	279	438	36%
1.0 - 1.5	654	920	29%	558	907	38%	113	231	51%
1.5 - 2.0	266	806	67%	229	759	70%	56	96	42%
2.0 - 3.0	204	623	67%	203	601	66%	59	90	34%
3.0 - 5.0	172	273	37%	150	264	43%	93	124	25%
> 5	182	237	23%	182	229	21%	113	144	22%
Total	2542	3940	35%	2323	3809	39%	713	1123	37%

6.4 Resaca de la Guerra

While the extent of the flooding around the resacas is not nearly of the same magnitude as what is experienced around the two drainage ditches, there are a couple of areas that are of concern as outlined in section 5.2.1. Currently the 100-yr floodplain covers an area of land approximately 842 acres in size, most of which is in developed portions of the watershed. However, it is important to note that that this acreage covers the resaca itself, streets, and land area just beyond the actual banks of the resaca that is still below the elevations of buildings in the region. Therefore, the number of structures impacted in this region is minimal compared to what it could be.

The problem within the RDLG watershed is minimal compared to that of the two main drainage ditches. Therefore only two sets of options were considered. These options (Option A & Option B) include runoff controls, detention ponds, property buyouts, and dredging. The specifics of each option are:

- Option A: Runoff controls on future development; 1, 50-acre detention pond; dredging downstream of Owen's Rd and property buyouts.
- Option B: Option A plus an additional 1000-cfs diversion near Quail Hollow and dredging from Laredo Rd. to Highway 77/83

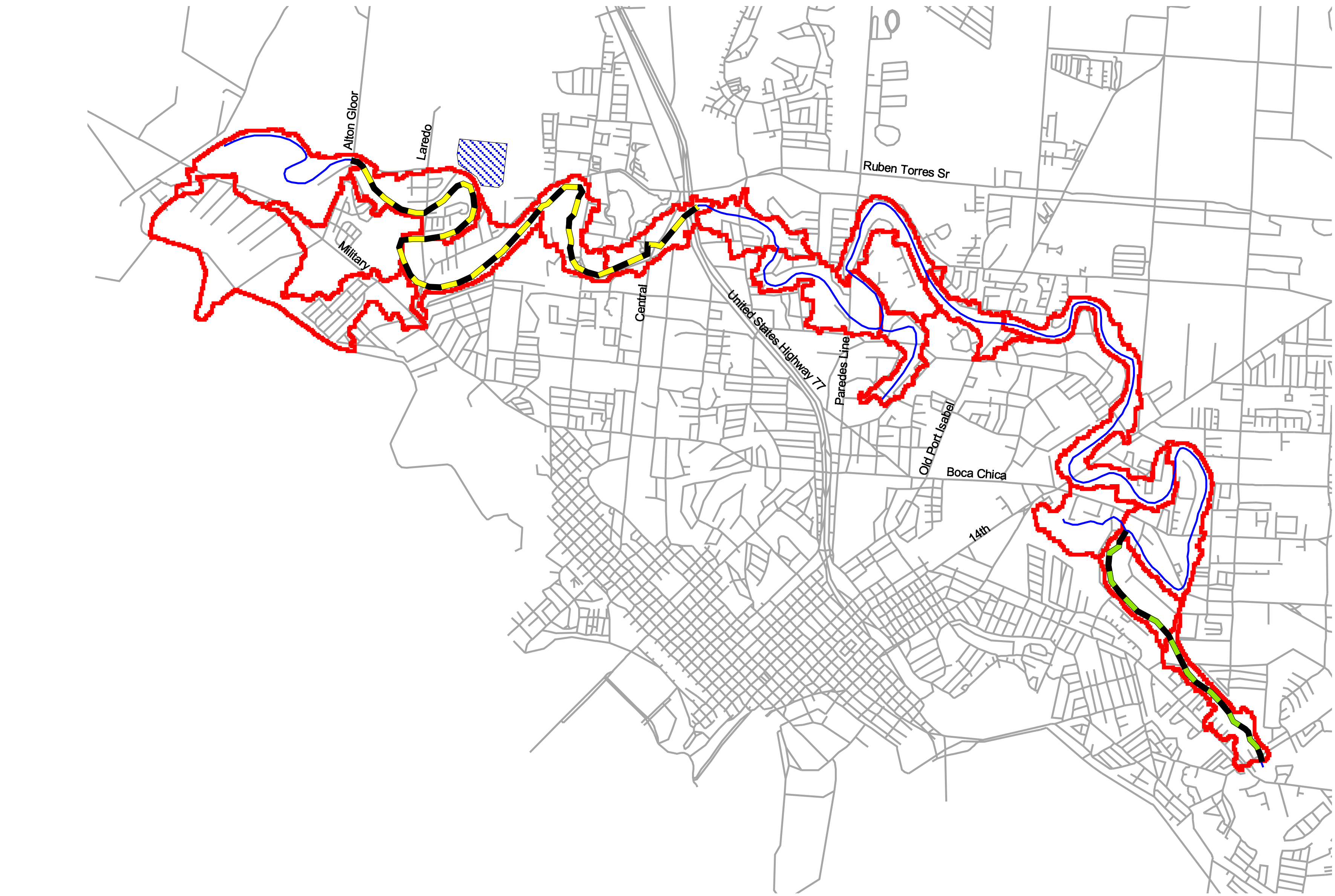
6.4.1 Option A

To address flooding in the uppermost portion of the watershed in the Quail Hollow area, a 50-acre detention pond is proposed (Figure 6-27). Because the area surrounding the resacas is elevated relative to the surrounding regions it will be necessary to pump water from the resaca into the detention pond to create relief for this area.

The area near Owen's Rd. that has experienced multiple flood events over the years will be bought out in this scenario thus removing the structures there from the floodplain.

The other problem areas within RDLG, which are discussed in section 5.2.1, will not be specifically addressed in this study. The impact that these regions have on surrounding structures is minimal in comparison to the problem at hand in CCDD1 and NMD, and few of the structures are within the floodplain.

Finally, dredging of the Resaca bottom is proposed in order to create extra storage in the downstream portion of RDLG for excess stormwater diverted from NMD (outlined in Option C-3) from Owens Rd. to the outfall into NMD (Figure 6-27). However, for this to be effective in creating excess storage in the Resaca, it will also be necessary to lower the height of the outfall structure going into NMD. This will maintain a lower water



RDLG Detention Pond
Dredge Location Option A
Dredge Location Option B
RDLG
RDLG Watershed
Streets

N

0 0.5 1 Miles

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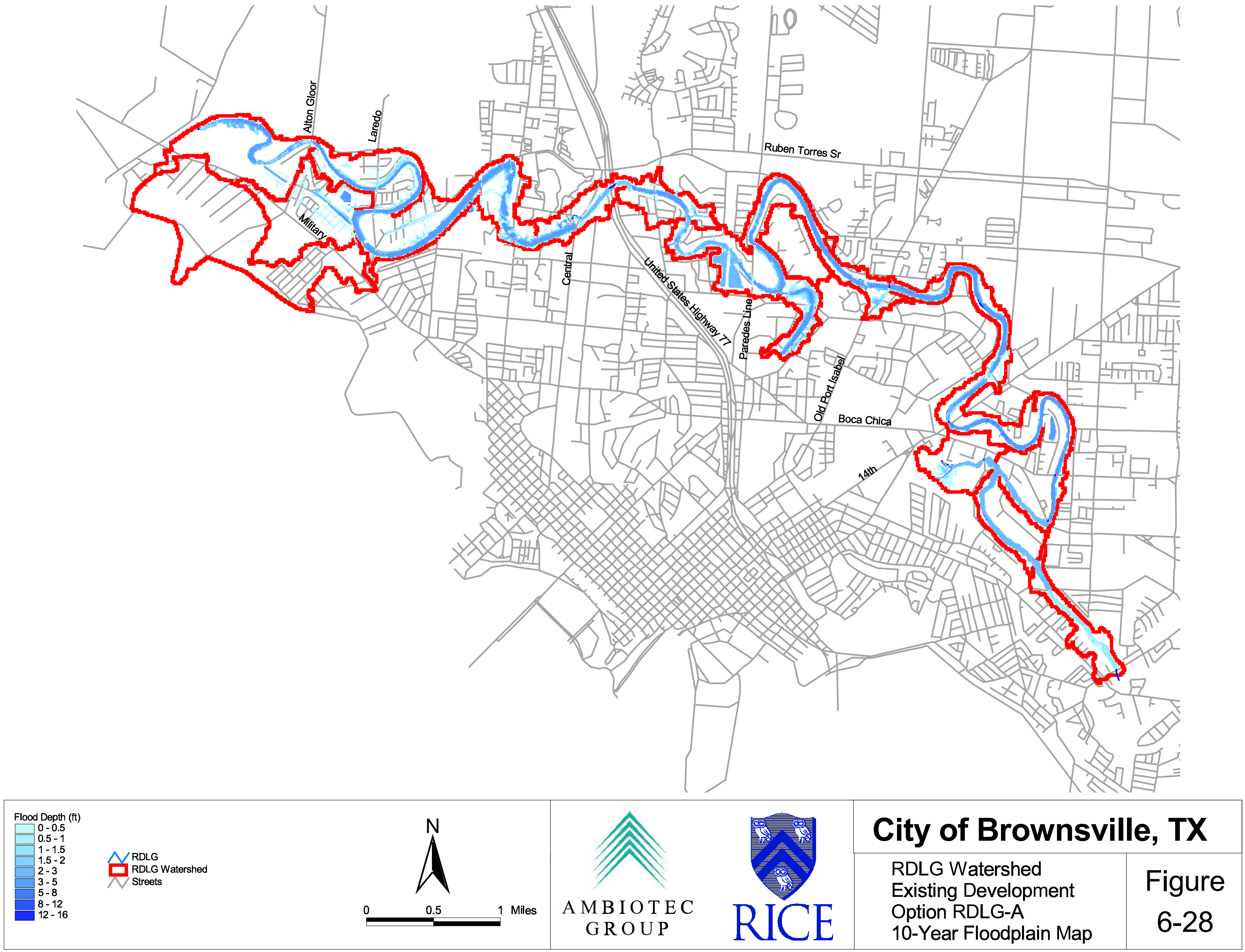
RDLG Watershed
Flood Protection Options

Figure
6-27

level in the resaca pools prior to a rainfall event and provide the added storage within the banks of the resaca. Dredging in this section of the resaca ranged from 1.5 to 4 feet depending on the estimated thickness of the sediment layer, and the drop structure at the outlet was lowered 2.5 feet.

The effects of the implementation of Option A in RDLG may be viewed in Tables 6-14 – 6-21. Overall, the extent of the floodplain is reduced by approximately 25% from the full development “no action” scenario and by 19% when Option A was implemented without runoff controls for the 100-yr event (11% and 7% respectively for the 10-yr event). This is accompanied by a 35% and 26% reduction in out of bank flooding volume (15% and 8% for the 10-yr). The number of total structures within the 100-yr floodplain has been reduced from 250 to 103 (a 59% reduction) relative to the full development no action conditions. However when the structural components of Option A are implemented without runoff controls, the number of flooded structures totaled 131 (a 48% reduction) illustrating the effectiveness of runoff controls on preventing the worsening of future flood damages. The 10-yr and 100-yr floodplains for Option A both with and without runoff controls may be viewed in Figures 6-28 – 6-31. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

Despite the reduction in floodplain area resulting from the proposed Option A improvements there exists one area where little to no improvement was observed. This is in the vicinity of the RDLG tributary near Owens Rd. This area is low relative to the surrounding watershed and has experienced many instances of flooding in recent years. Therefore it is recommended that a berm be built in this region to protect these homes from potential flooding.



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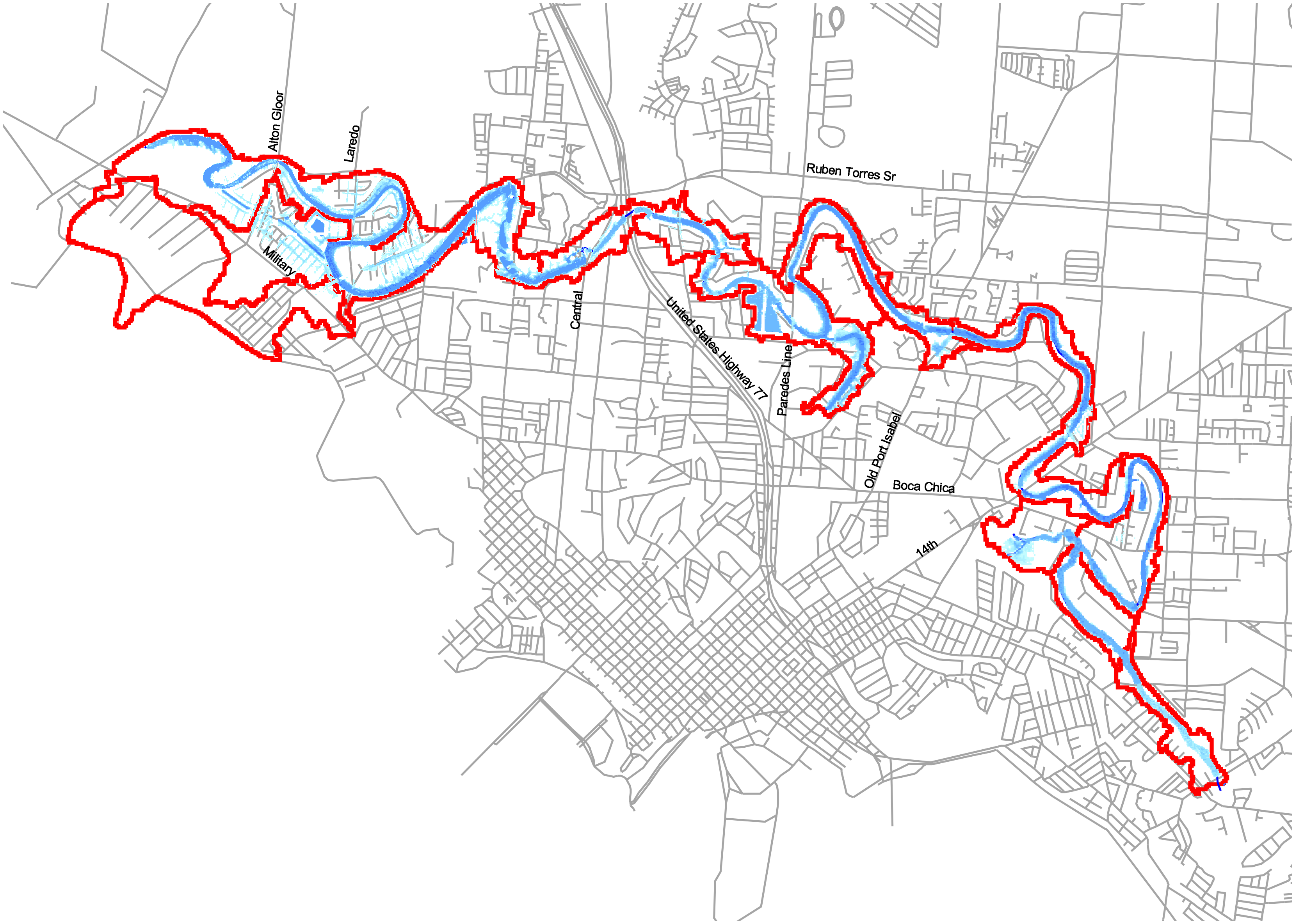
RDLG Watershed
Existing Development
Option RDLG-A
10-Year Floodplain Map

Figure 6-28



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

RDLG
RDLG Watershed
Streets

N

0 0.5 1 Miles

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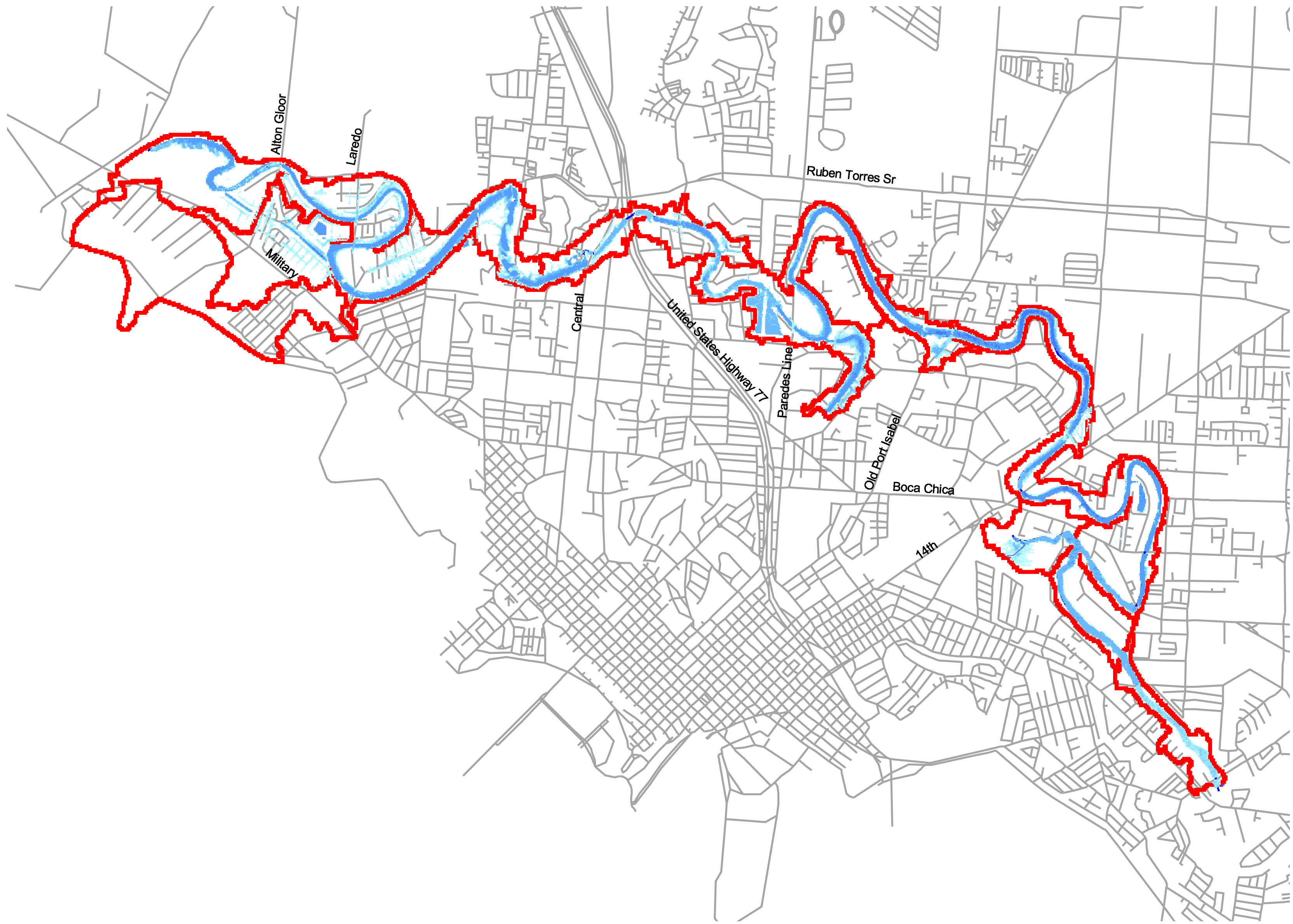
RDLG Watershed
Existing Development
Option RDLG-A
100-Year Floodplain Map

Figure 6-29



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

RDLG
RDLG Watershed
Streets

N

0 0.5 1 Miles

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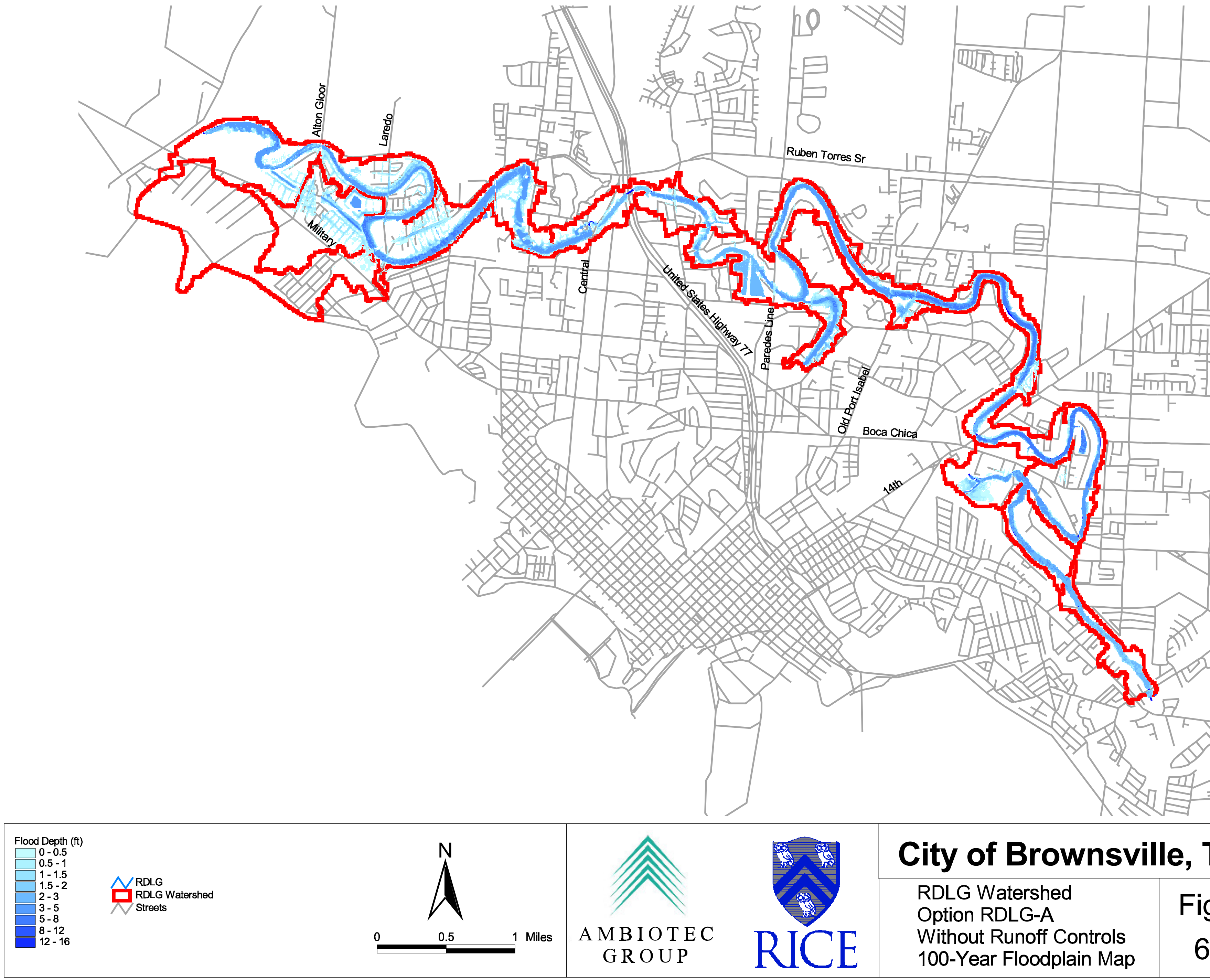
RDLG Watershed
Option RDLG-A
Without Runoff Controls
10-Year Floodplain Map

Figure 6-30



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RDLG Watershed
Option RDLG-A
Without Runoff Controls
100-Year Floodplain Map

Figure 6-31

In regions where a berm is ineffective or unpractical, these structures should be considered for a potential buy-out option.

6.4.2 Option B

To further mitigate the flooding problem in the Quail Hollow subdivision, Option B includes two, 1000-cfs diversions in the upstream portion of the watershed. In addition to these diversions the sediment layer at the bottom of the resaca throughout Quail Hollow and the Valley International Country Club (VICC) will be dredged. Throughout the dredged area, the 2 weir structures that are currently in place were dropped by 2-ft.

The effects of implementing Option B may be viewed in Tables 6-14 – 6-20 and Figures 6-32 – 6-35. Overall, the extent of the 100-yr floodplain was reduced by 36% (10-yr by 18%) relative to the full development no action scenario. Out of bank flooding volumes for the 100-yr event was reduced by 45% relative to the full development no action scenario (21% for the 10-yr). In terms of flooded structures, nearly a 70% reduction was observed during a 100-yr event (31% for the 10-yr) as the number of impacted buildings fell from 250 to 78.

The effects of implementing the structural components of Option B without runoff controls on future development resulted in a reduction of floodplain area of 34% (16% for the 10-yr)(Table 6-20). Out of bank flooding volume was reduced by 42% (19% for the 10-yr) and the number of structures within the floodplain was reduced by 67% (29% for the 10-yr). Table 6-21 reveals that there is only a 5% reduction in the total number of flooded structures for a 100-yr event when Option B is implemented with runoff controls versus without. Unlike that of CCDD1 the implementation of runoff controls plays only a small roll in the effectiveness of the various flood mitigation options. This is most likely due to the level of development that

Table 6-14. RDLG Water Surface Elevations after Implementation of Options A & B

Location	Cross Section	Profile	Water Surface Elevation (ft)			
			Existing No Action	Full No Action	Option A	Option B
Alton Gloor	84060	10 year 100 year	33.74 34.53	33.98 34.87	33.45 33.97	33 33.12
Laredo	75700	10 year 100 year	33.69 34.47	33.85 34.7	33.33 33.85	32.96 32.96
Golf Course (entering)	67600	10 year 100 year	33.12 33.81	33.36 34.13	33.06 33.37	32.68 32.76
Golf Cart Bridge	60700	10 year 100 year	32.69 32.98	32.72 33.17	32.77 32.81	32.65 32.66
Central Blvd	59700	10 year 100 year	32.64 32.84	32.63 33	32.63 32.68	32.64 32.64
US 83/77	57800	10 year 100 year	31.37 31.8	31.54 32.03	31.66 31.68	30.82 31.13
Old Alice	56200	10 year 100 year	31.08 31.42	31.22 31.56	31.43 31.45	30.16 30.77
Paredes Line	49300	10 year 100 year	29.31 29.33	29.23 29.36	29.41 29.49	29.82 29.85
Old Port Isabel	36150	10 year 100 year	28.04 28.39	28.45 28.77	28.1 28.32	28.08 28.46
Price Rd	27300	10 year 100 year	27.5 27.87	27.89 28.11	27.59 27.83	27.52 27.92
Padre Island Highway	24600	10 year 100 year	26.19 26.3	26.23 26.37	26.22 26.39	26.01 26.15
Boca Chica	15300	10 year 100 year	24.33 24.66	24.45 25.28	24.35 24.76	23.98 24.23
Billy Mitchell Blvd	13300	10 year 100 year	23.72 23.88	23.81 24.92	23.68 23.87	23.56 23.67
Acacia Lake Drive	11000	10 year 100 year	22.65 23.14	23.09 24.85	22.36 22.68	22.25 22.62
Morningside Rd	200	10 year 100 year	21.08 21.52	22.09 24.29	20.75 21.05	20.66 21
Outlet	55	10 year 100 year	15.91 16.17	21.99 24.22	13.42 13.62	13.35 13.6

currently exists in each of the respective watersheds and the fact that there is a much larger percentage of undeveloped area in CCDD1 than RDLG.

Table 6-15. RDLG Out of Bank Flooding Volume and Floodplain Area After Implementation of Options A & B

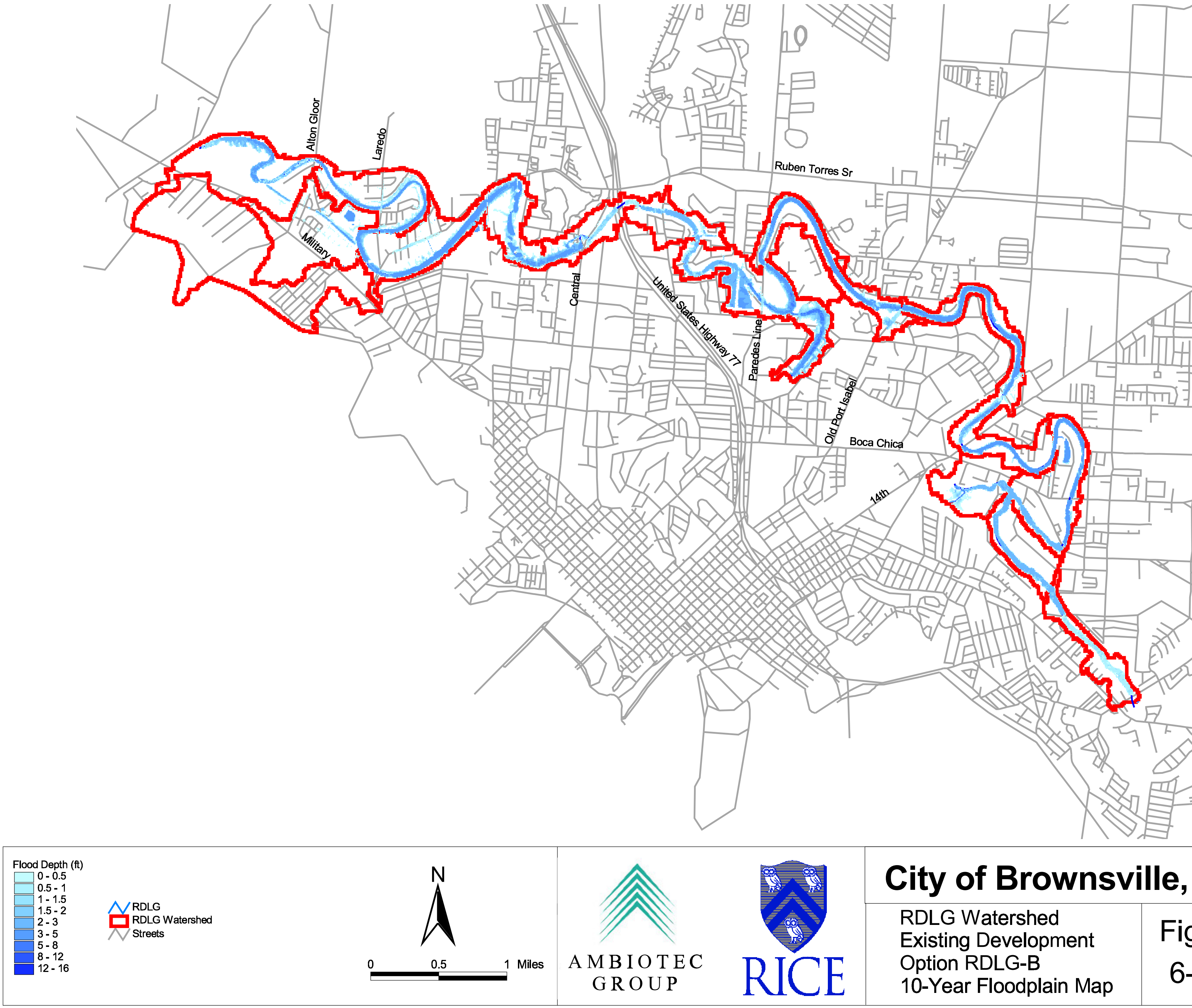
	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	**Area of Floodplain (acre)**	**Total Out of Bank Volume (ac-ft)**	**Area of Floodplain (acre)**
Existing	678	675	926	842
Full Development	762	725	1196	970
Option A	651	646	776	723
Option B	604	596	657	624

Table 6-16. RDLG Number of Flooded Structures for the 10-yr Event After Implementation of Options A & B

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A	Option B
0.5 - 1.0	37	45	34	26
1.0 - 1.5	18	17	15	16
1.5 - 2.0	8	17	5	9
2.0 - 3.0	15	15	17	13
3.0 - 5.0	4	7	5	6
> 5	0	1	0	0
Total	82	102	76	70

Table 6-17. RDLG Number of Flooded Structures for the 100-yr Event After Implementation of Options A & B

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A	Option B
0.5 - 1.0	61	105	44	27
1.0 - 1.5	37	70	20	18
1.5 - 2.0	17	26	13	11
2.0 - 3.0	27	31	19	13
3.0 - 5.0	6	17	6	9
> 5	1	1	1	0
Total	149	250	103	78



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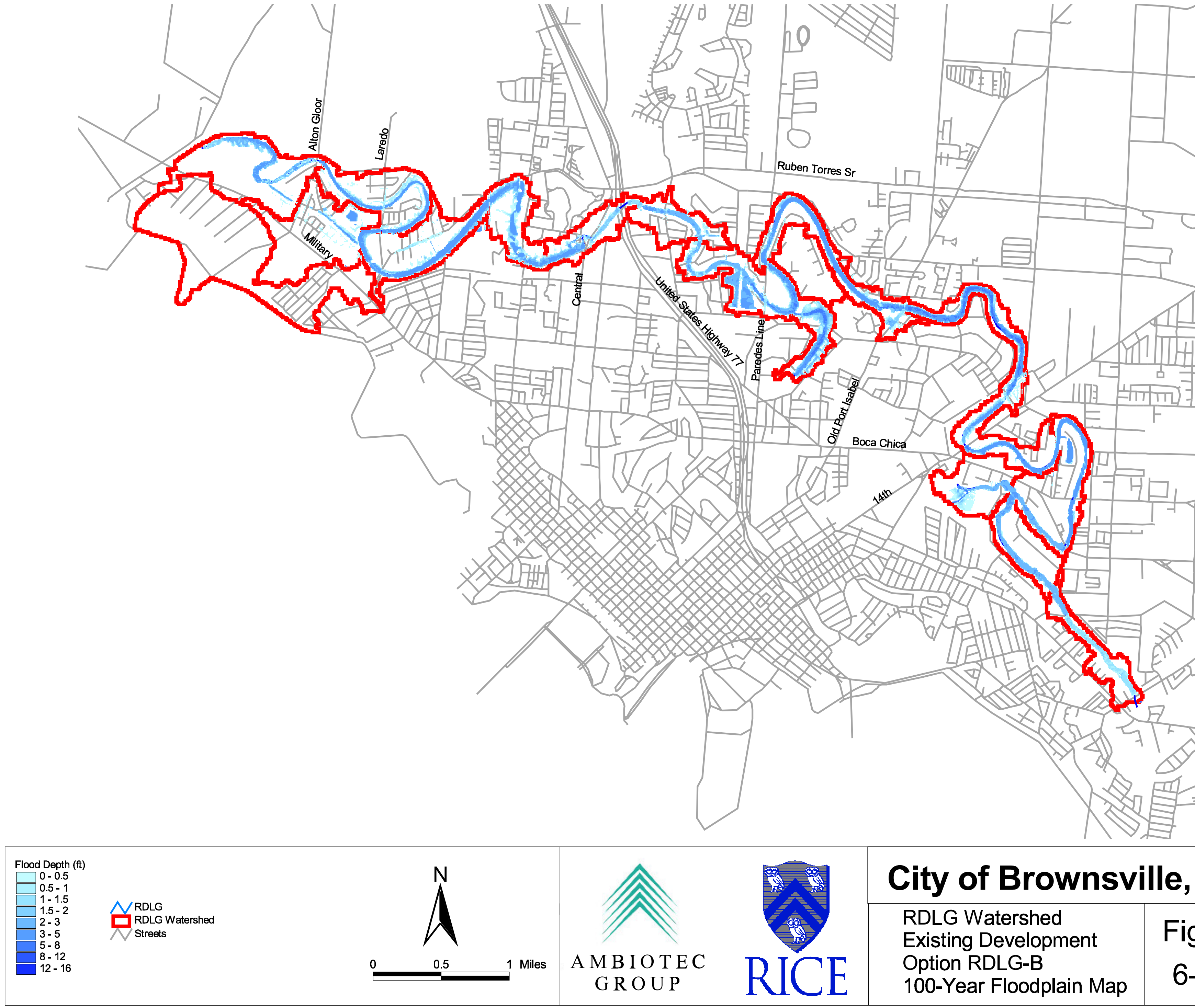
RDLG Watershed
Existing Development
Option RDLG-B
10-Year Floodplain Map

Figure 6-32



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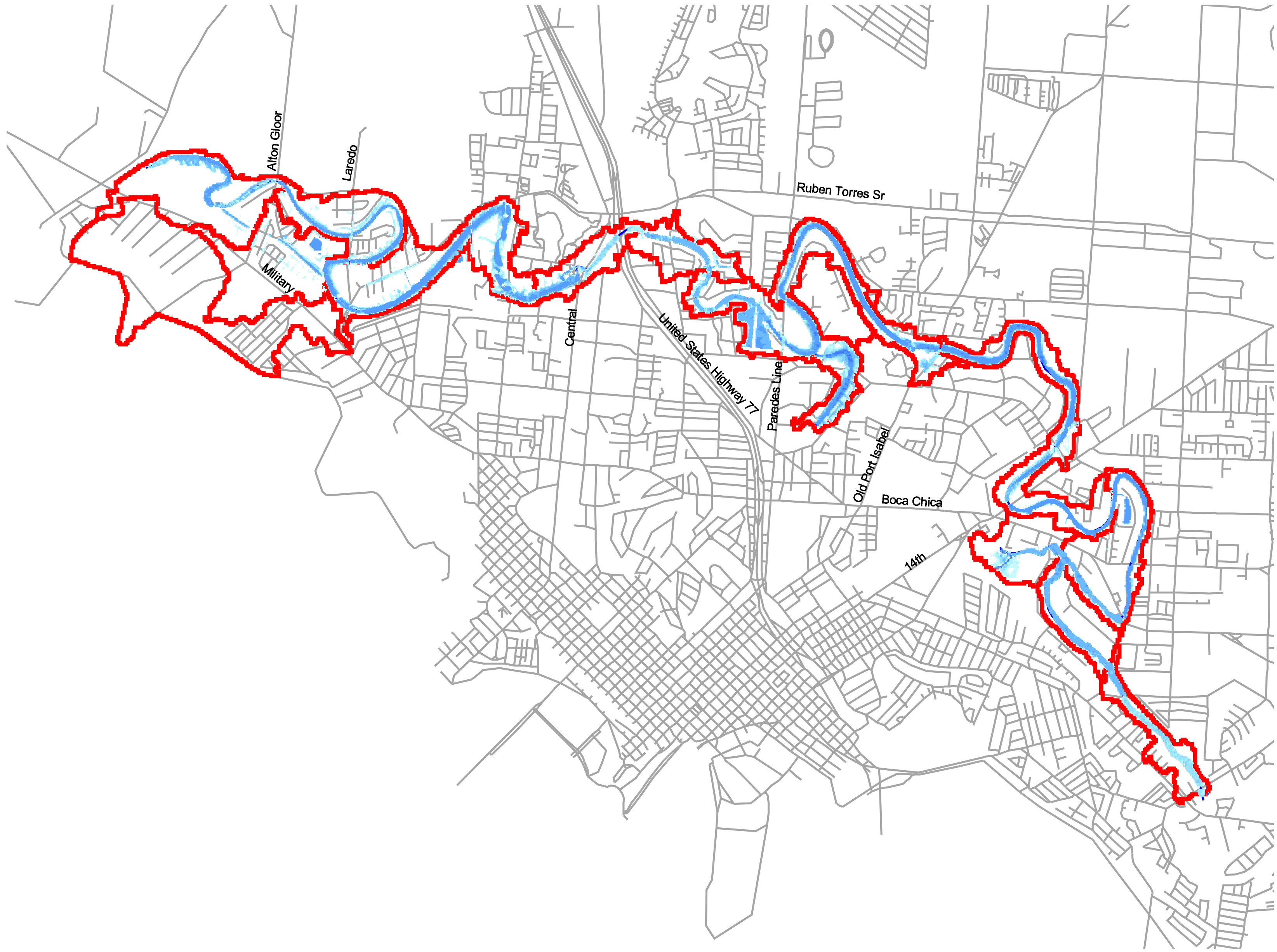
RDLG Watershed
Existing Development
Option RDLG-B
100-Year Floodplain Map

Figure 6-33



Flood Depth (ft)
0 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 5
5 - 8
8 - 12
12 - 16

RDLG
RDLG Watershed
Streets

N

0 0.5 1 Miles

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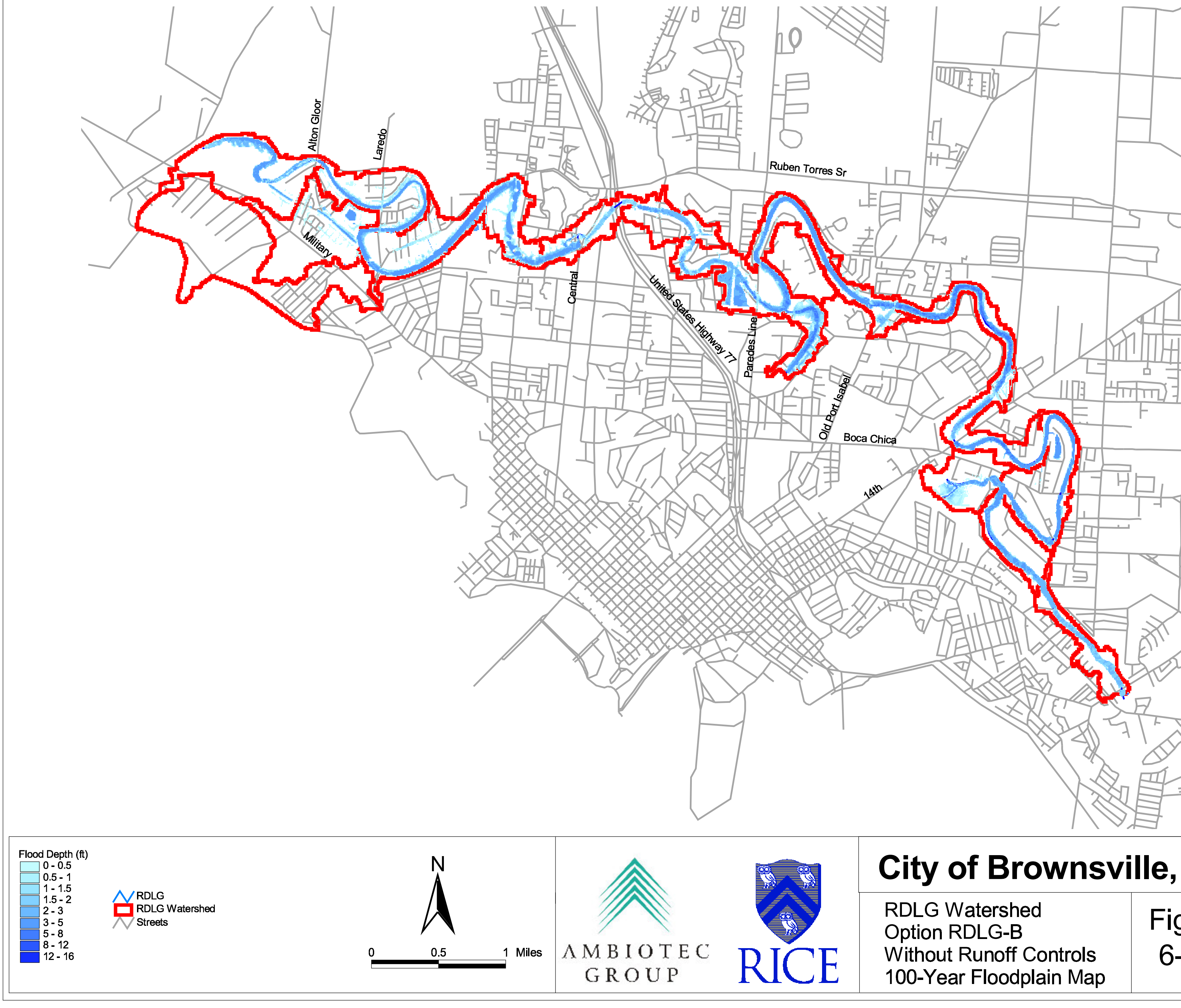
RDLG Watershed
Option RDLG-B
Without Runoff Controls
10-Year Floodplain Map

Figure 6-34



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RDLG Watershed
Option RDLG-B
Without Runoff Controls
100-Year Floodplain Map

Figure 6-35

Table 6-18. RDLG Water Surface Elevations after Implementation of Options A & B Without Runoff Controls

Location	Cross Section	Profile	Water Surface Elevation (ft) Existing	Full Development	Option A	Option B
Alton Gloor	84060	10 year 100 year	33.74 34.53	33.98 34.87	33.64 34.31	33.12 33.31
Laredo	75700	10 year 100 year	33.69 34.47	33.85 34.7	33.43 34.01	32.96 33.05
Golf Course (entering)	67600	10 year 100 year	33.12 33.81	33.36 34.13	33.1 33.54	32.7 32.76
Golf Cart Bridge	60700	10 year 100 year	32.69 32.98	32.72 33.17	32.74 32.86	32.65 32.65
Central Blvd	59700	10 year 100 year	32.64 32.84	32.63 33	32.63 32.73	32.64 32.64
US 83/77	57800	10 year 100 year	31.37 31.8	31.54 32.03	31.65 31.74	30.84 31.17
Old Alice	56200	10 year 100 year	31.08 31.42	31.22 31.56	31.36 31.38	30.2 30.87
Paredes Line	49300	10 year 100 year	29.31 29.33	29.23 29.36	29.34 29.51	29.77 29.86
Old Port Isabel	36150	10 year 100 year	28.04 28.39	28.45 28.77	28.52 28.87	28.13 28.43
Price Rd	27300	10 year 100 year	27.5 27.87	27.89 28.11	27.94 28.17	27.56 27.91
Padre Island Highway	24600	10 year 100 year	26.19 26.3	26.23 26.37	26.26 26.5	26.05 26.23
Boca Chica	15300	10 year 100 year	24.33 24.66	24.45 25.28	24.46 25	24.03 24.4
Billy Mitchell Blvd	13300	10 year 100 year	23.72 23.88	23.81 24.92	23.73 24	23.6 23.75
Acacia Lake Drive	11000	10 year 100 year	22.65 23.14	23.09 24.85	22.54 23.01	22.48 22.94
Morningside Rd	200	10 year 100 year	21.08 21.52	22.09 24.29	21.14 21.83	21.12 21.77
Outlet	55	10 year 100 year	15.91 16.17	21.99 24.22	19.21 21.19	18.87 21.14

Table 6-19. RDLG Out of Bank Flooding Volume and Floodplain Area After Implementation of Options A & B Without Runoff Controls

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	678	675	926	842
Full Development	762	725	1196	970
Option A	700	673	887	786
Option B	615	606	688	643

Table 6-20. RDLG Number of Flooded Structures After Implementation of Options A & B With and Without Runoff Controls During a 100-Year Event

Water Depth (ft)	Option A	Option A w/o Runoff Controls	% Change	Option B	Option B w/o Runoff Controls	% Change
0.5 - 1.0	44	52	15%	27	28	4%
1.0 - 1.5	20	32	38%	18	17	-6%
1.5 - 2.0	13	22	41%	11	14	21%
2.0 - 3.0	19	13	-46%	13	13	0%
3.0 - 5.0	6	10	40%	9	10	10%
> 5	1	2	50%	0	0	0%
Total	103	131	21%	78	82	5%

6.5 Town Resaca

While the 100-yr floodplain for TR is smaller than that of RDLG, 494 acres verses 842 acres, the largest portion of the floodplain in TR lies over a heavily developed area thus impacting more structures and inflicting more damage. There are three main areas of concern in this watershed. The main areas of concern in this watershed are the segments prior and after the Gladys Porter Zoo in the upper middle portion of the watershed. One in particular, located on the western side of the resaca along Ringgold Street, is relatively low compared to the surrounding region.

The second area is located just south of the zoo between 4th St. and 10th St. and is the largest portion of the floodplain. This area experiences street flooding up to 3 feet deep for the 100-yr event with overland flooding approaching depths of 2 feet. The last major area of concern is near the outfall into NMD. Under the current land use conditions only street flooding is observed for the 100-yr event. However as the region approaches the full development scenario, this residential area becomes completely inundated with up to 3.5 feet of water. The proposed options include dredging of the Resaca segment before and after the zoo; buyouts of the area near Ringgold Street; widening and lining of Resaca segment near the Impala Pump station that connects with the NMD; and improving the Impala Pump station by expanding the sump area and increasing the pumping capacity to approximately 1000 cfs.

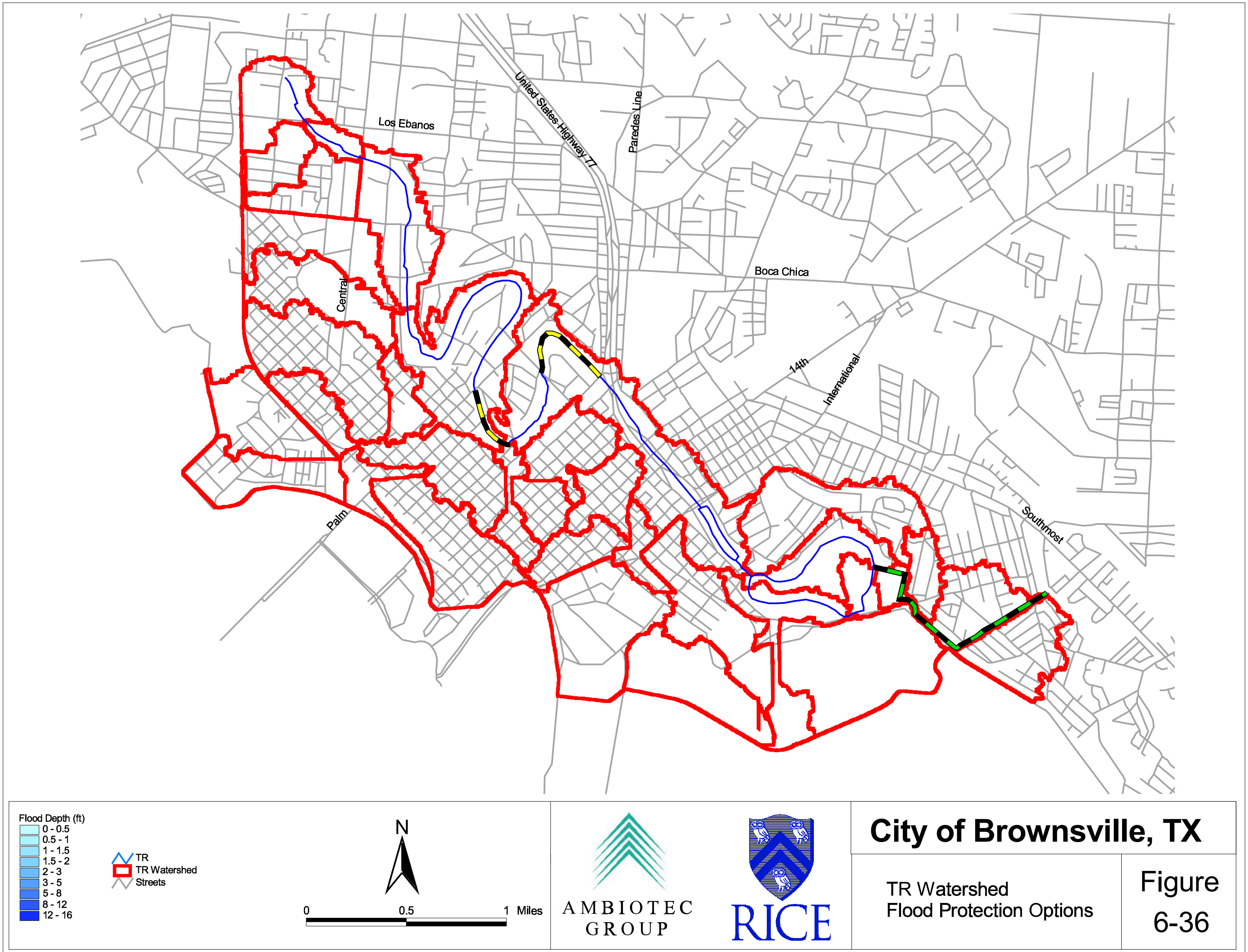
6.5.1 Option A

The proposed options are shown in Figure 6-36 and include implementing runoff controls on future developments. The aim of the proposed dredging is to create extra storage within the resaca and provide relief for the areas before and after the zoo. However, it is again important to note that to create additional storage in these areas it is necessary to lower the static water level in these pools permanently by lowering weir heights. Overall the areas entering and exiting the zoo were dredged between 3 and 4 feet in the model and the controlling weir structures were lowered 1.5 feet.

The Impala Pump Station upgrade would include increasing the pumping capacity of the station, enlarging the sump area to improve pumping efficiency, and the lining of the ditch south of the Brownsville PUB Southside waste water treatment plant leading to the pump station. The effects of lining the ditch are included in Option A along with the proposed dredging before and after the zoo.

The effects of the implementation of Option A in TR may be viewed in Tables 6-21 - 6-25. Overall, the extent of the floodplain is reduced by approximately 37% from the full development no action scenario and only 28% when runoff controls are not implemented for the 100-yr event. This is accompanied by a 43% and 28% reduction in out of bank flooding volume respectively. The number of total structures within the 100-yr floodplain has been reduced from 899 to 201 relative to the full development conditions. When Option A is implemented without runoff controls the number of impacted structures is 279. The 10-yr and 100-yr floodplains for Option A with and without runoff controls may be viewed in Figures 6-37 – 6-40. Floodplains for the 2-yr, 5-yr, 25-yr, and 50-yr events may be viewed in Appendix C along with the water surface profiles for each event.

While there was a noted improvement observed after the proposed improvements were modeled in the region, the area exiting the zoo is still plagued by flooding. This is another area where in recent years several instances of flooding have been observed. For this reason there are 3 to 5 structures in this region that have experienced repeated instances of flooding that are recommended as potential candidates for buy-out options.



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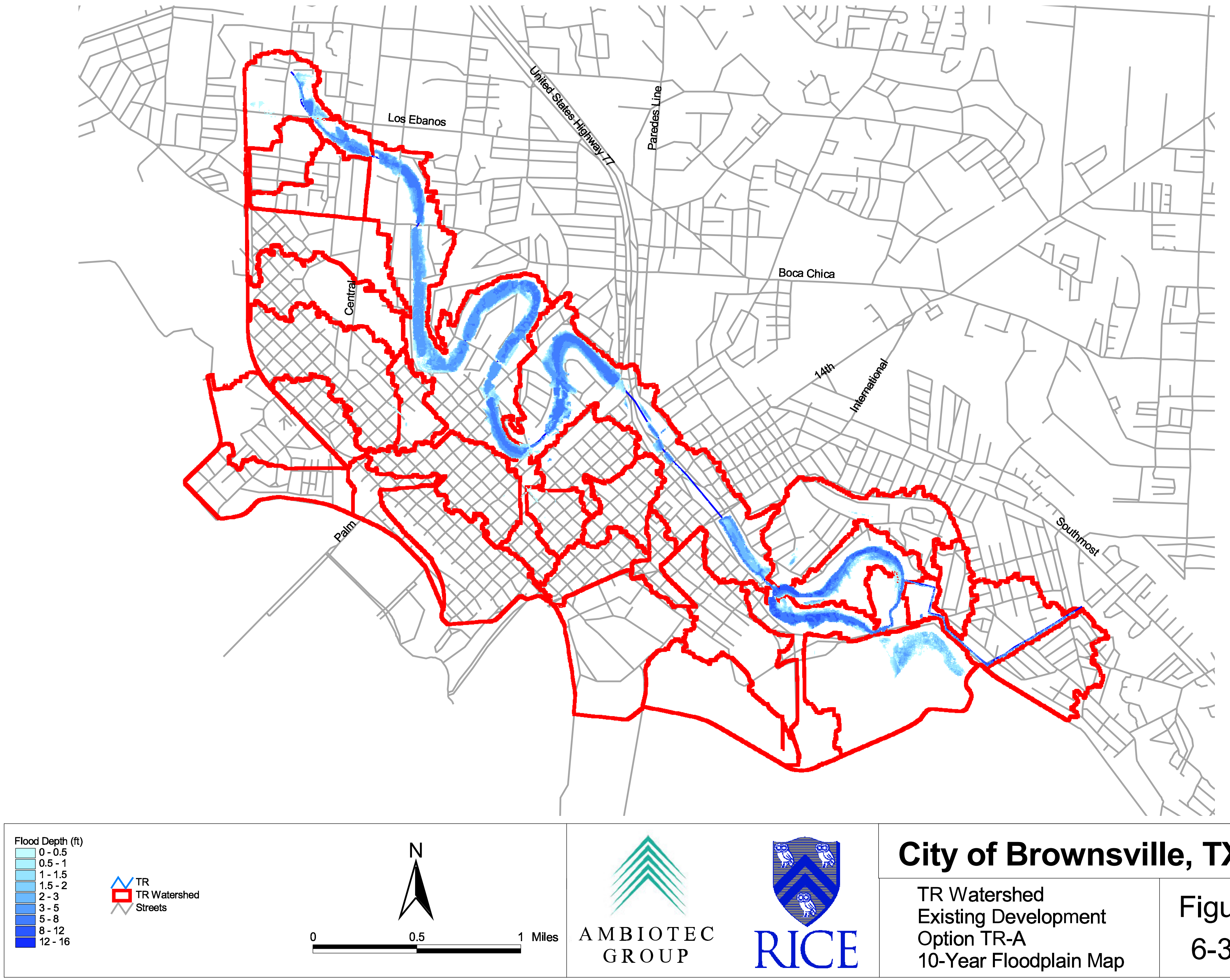
TR Watershed
Flood Protection Options

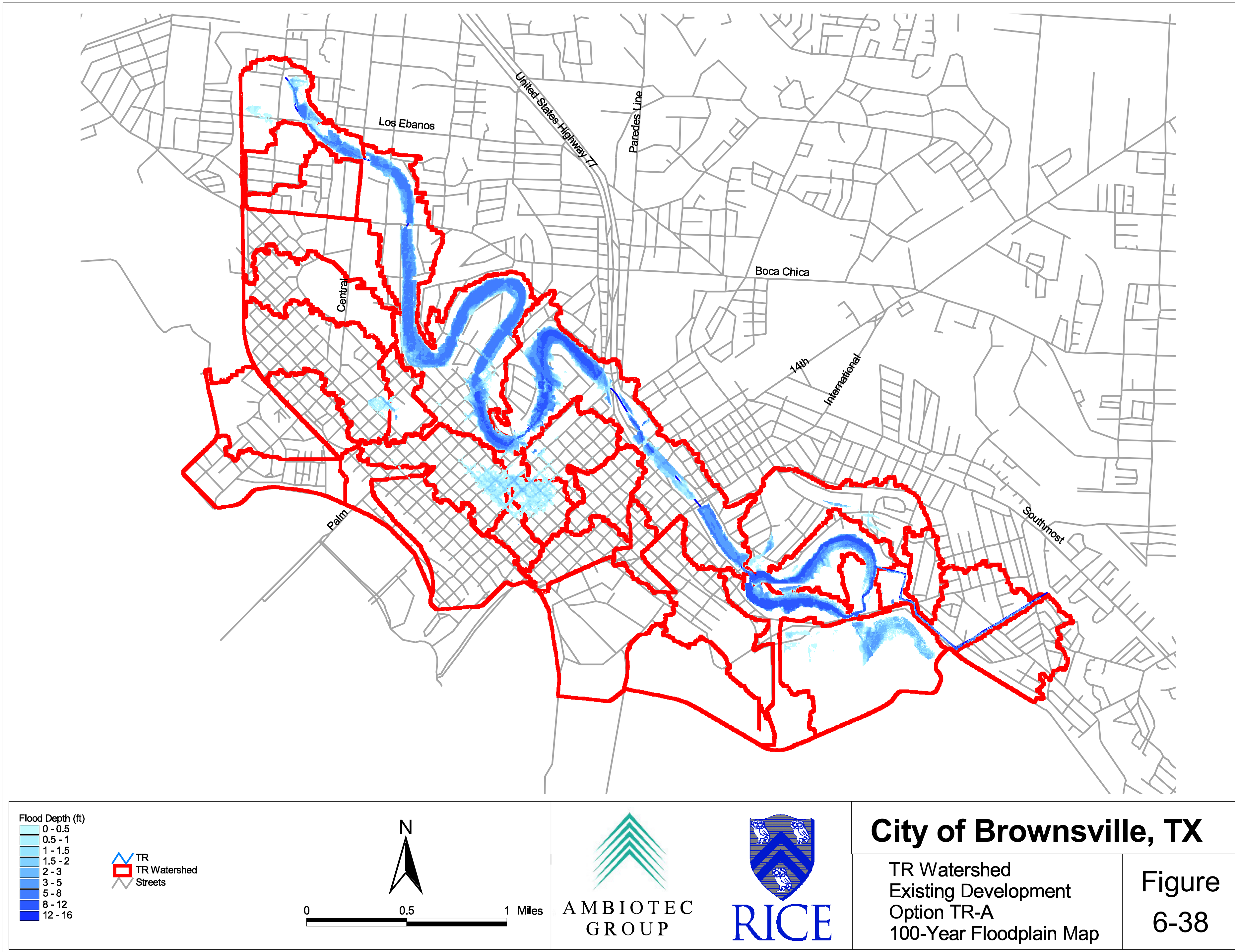
Figure 6-36



City of Brownsville, TX

TR Watershed
Existing Development
Option TR-A
10-Year Floodplain Map

Figure 6-37



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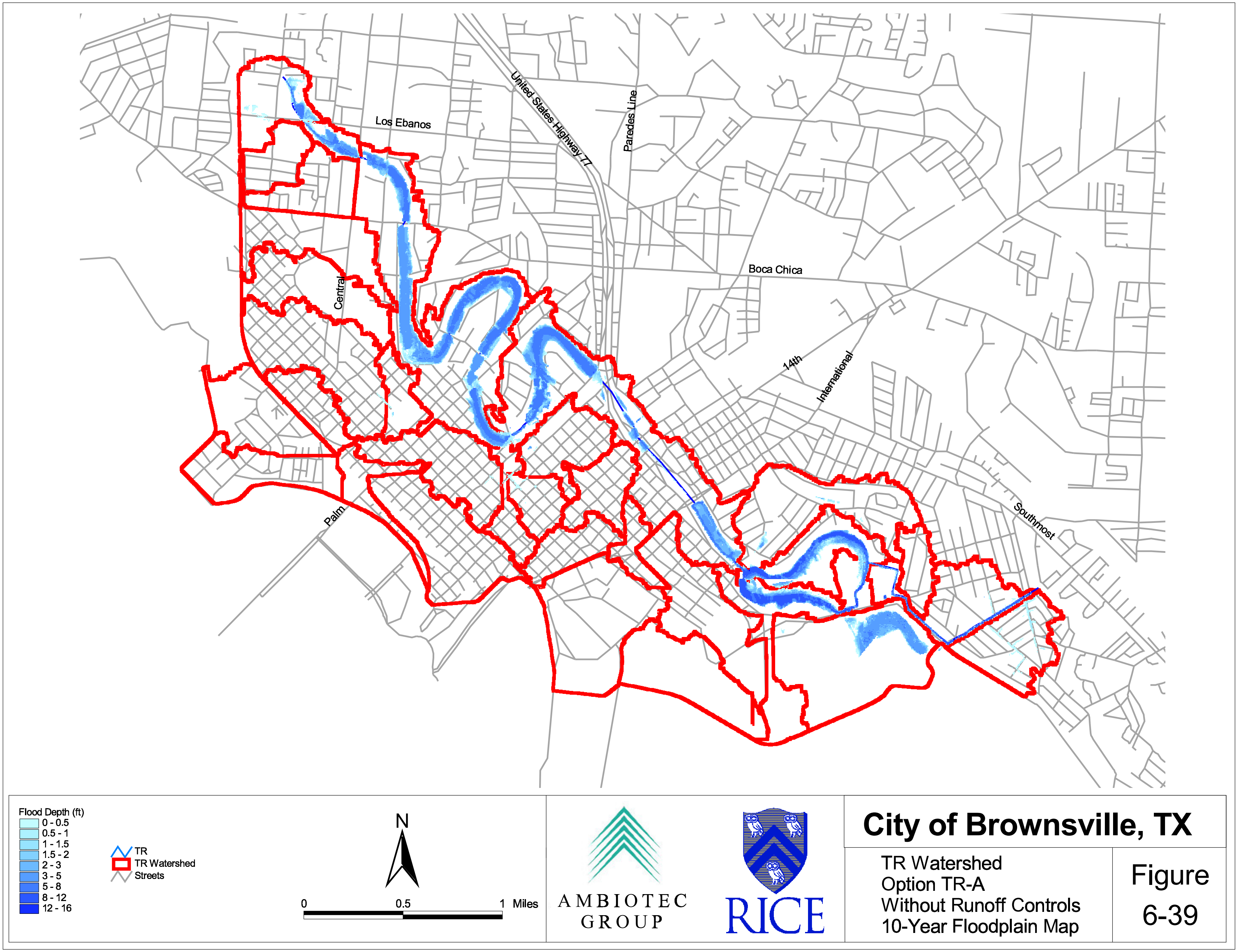
TR Watershed
Existing Development
Option TR-A
100-Year Floodplain Map

Figure 6-38



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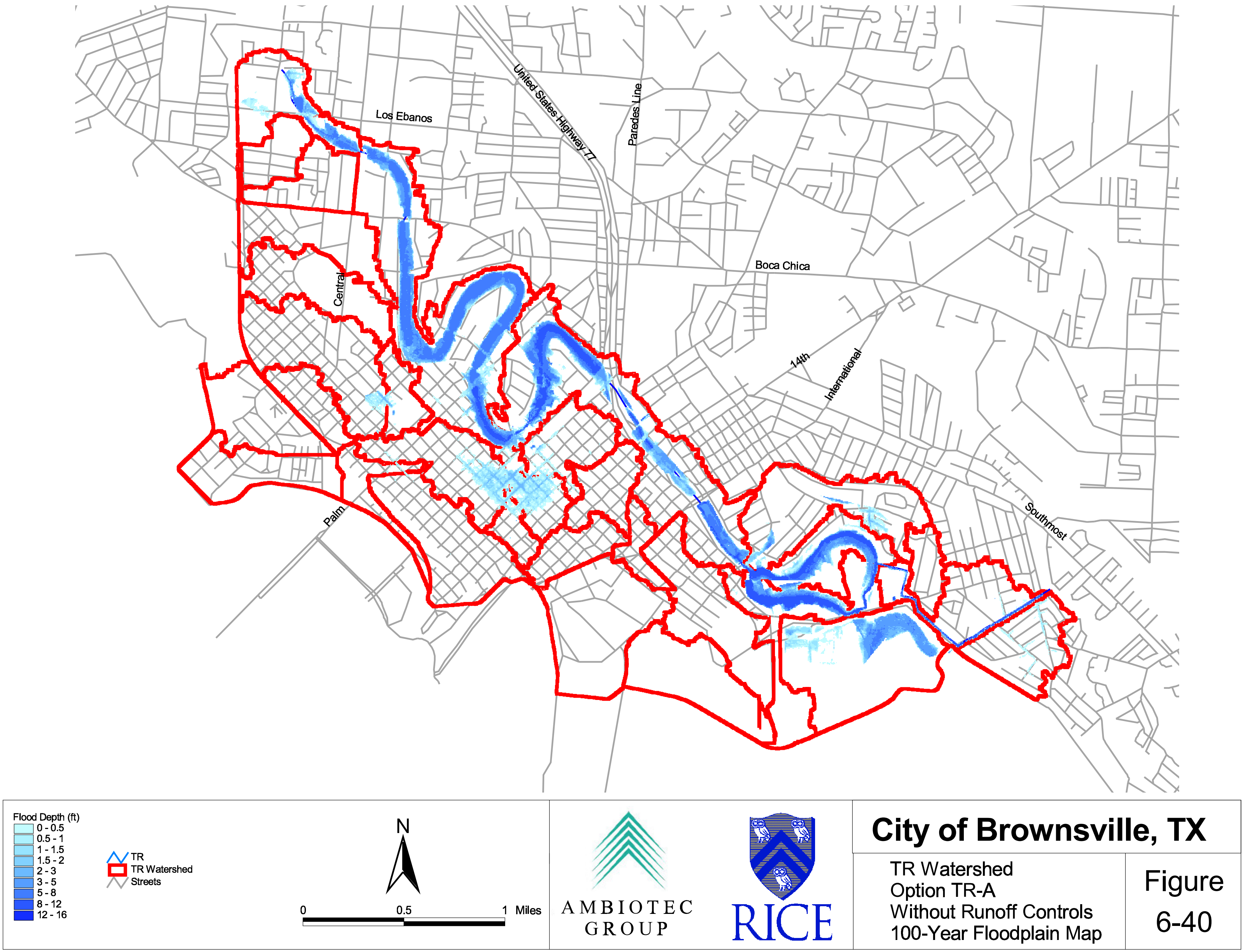
TR Watershed
Option TR-A
Without Runoff Controls
10-Year Floodplain Map

Figure 6-39



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TR Watershed
Option TR-A
Without Runoff Controls
100-Year Floodplain Map

Figure 6-40

Table 6-21. TR Water Surface Elevations After Implementation of Option A

				Water Surface Elevation (ft)		
Location	**Cross Section**	**Profile**	**Q Total**	**Existing**	**Full Development**	**Option A**
Los Ebanos Blvd.	39450	10	46	31.57	31.48	31.45
		100	68	31.85	32.07	31.9
Central Blvd.	37600	10	164	31.56	31.61	31.43
		100	338	31.84	32.03	31.86
Boca Chica Blvd.	35150	10	111	31.51	31.6	31.42
		100	371	31.79	31.95	31.84
Belthair St.	33700	10	68	29.55	30.28	28.98
		100	308	31.33	31.4	30.93
Calle Retama	30300	10	68	29.52	30.27	28.95
		100	308	31.31	31.38	30.92
Calle Retama	25500	10	210	29.38	30.12	28.89
		100	316	31.29	31.37	30.78
Palm Blvd.	24700	10	210	29.25	29.95	28.71
		100	316	31.28	31.35	30.78
Old Alice	22600	10	311	29.24	29.94	28.69
		100	385	31.27	31.34	30.76
Ringgold St.	20300	10	838	28.51	29.37	27.71
		100	1211	30.5	30.65	29.76
US 83/77	17200	10	820	28.48	29.34	27.69
		100	1162	30.47	30.62	29.74
12th St.	15400	10	738	28.33	29.25	27.4
		100	1061	30.35	30.5	29.58
International Blvd	11000	10	738	27.36	28.24	25.42
		100	1054	29.04	29.51	27.41
East Ave.	4500	10	1102	25.62	27.15	22.43
		100	1182	27.55	27.87	22.7
Impala Drive	2600	10	1110	25.39	27.05	21.96
		100	1199	27.38	27.71	22.25
Calle Milpa Verde Dr.	1700	10	1110	23.98	26.55	20.42
		100	1199	25.8	27.02	20.67
Tulipan	400	10	1110	21.98	26.46	19.47
		100	1199	24.28	26.94	19.72
Outlet	213	10	1763	16.8	26.42	18.2
		100	1764	17.35	26.9	18.2

Table 6-22. TR Out of Bank Flooding Volume and Floodplain Area After Implementation of Option A

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)	Total Out of Bank Volume (ac-ft)	Area of Floodplain (acre)
Existing	567	319	1032	494
Full Development	840	456	1245	617
Option A	346	270	708	387

Table 6-23. TR Number of Flooded Structures for the 10-yr Event After Implementation of Option A

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A
0.5 - 1.0	10	206	16
1.0 - 1.5	17	99	13
1.5 - 2.0	14	19	14
2.0 - 3.0	27	29	22
3.0 - 5.0	29	48	14
> 5	9	14	8
Total	106	415	87

Table 6-24. TR Number of Flooded Structures for the 100-yr Event After Implementation of Option A

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A
0.5 - 1.0	139	352	71
1.0 - 1.5	88	296	20
1.5 - 2.0	17	114	13
2.0 - 3.0	26	41	28
3.0 - 5.0	56	57	47
> 5	36	39	22
Total	362	899	201

Table 6-25. TR Out of Bank Flooding Volume and Floodplain Area Without Runoff Controls

	10-year Design Storm		100-year Design Storm	
	Total Out of Bank Volume (ac-ft)	**Area of Floodplain (acre)**	**Total Out of Bank Volume (ac-ft)**	**Area of Floodplain (acre)**
Existing	567	319	1032	494
Full Development	840	456	1245	617
Option A	447	298	895	445

Table 6-26. TR Number of Flooded Structures for the 10-yr Event After Implementation of Option A Without Runoff Controls

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A
0.5 - 1.0	10	206	14
1.0 - 1.5	17	99	14
1.5 - 2.0	14	19	18
2.0 - 3.0	27	29	23
3.0 - 5.0	29	48	21
> 5	9	14	9
Total	106	415	99

Table 6-27. TR Number of Flooded Structures for the 100-yr Event After Implementation of Option A Without Runoff Controls

Water Depth (ft)	Existing No Modifications	Full No Modifications	Option A
0.5 - 1.0	139	352	105
1.0 - 1.5	88	296	51
1.5 - 2.0	17	114	13
2.0 - 3.0	26	41	28
3.0 - 5.0	56	57	55
> 5	36	39	27
Total	362	899	279

6.6 Proposed Capital Improvements

6.6.1 Construction Costs

The capital costs associated with the implementation of all proposed options in Sections 6.0-6.5 totals approximately $202 million. This figure includes the costs of construction, engineering, land acquisition and property buy-outs. Administrative costs, including those associated with the development ordinances, development controls and agency reorganizations, are not included in this total as they are significantly lower than the capital costs. As shown in Table 6-23, the total capital cost includes approximately $45 million in improvements for the North Main Drain, $128 million for CCDD1, $20 million for Resaca de la Guerra, and $9 million for Town Resaca. Detailed breakdowns of the capital costs by alternatives within each of the options for each the four watersheds are presented in Tables 6-24 through 6-27. The overall cost effectiveness of each option will be further examined in section 6.6.2.

Table 6-28. Capital Costs for the Implementation of all Proposed Options

Watershed	Capital Costs
NMD	$45,117,190
CCDD1	$130,779,730
RDLG	$20,285,000
TR	$9,130,000
Total	**$205,311,920**

Table 6-29. Estimated Construction Costs of Proposed NMD Flood Mitigation Projects

North Main Drain Improvements	Costs
OPTION A	
Construct Coria Detention Pond	$604,200
Construct Price Road Detention Pond	$855,600
Construct Old Port Isabel Rd Detention Pond	$5,791,500
Construct City Detention Pond Near Owens Road	$2,035,500
Construct City Detention Pond Near Airport	$4,036,500
Construct Levee around southern portion of airport	$567,190
OPTION A TOTALS:	**$13,890,490**
OPTION B	
Line ditch to top of bank from 77/83 to RDLG confluence	$15,990,000
OPTION B TOTALS:	**$15,990,000**
OPTION C.1	
Line ditch to top of bank from 77/83 to airport	$3,997,500
Re-grade ditch to divert 1000 cfs of flow from US 83/77 westward	$1,995,480
OPTION C.1 TOTALS:	**$5,992,980**
OPTION C.2	
Re-grade ditch to divert 2500 cfs of flow from US 83/77 westward	$2,993,220
OPTION C.2 TOTALS:	**$2,993,220**
OPTION C.3	
Construct Pumping Station to Resaca de la Guerra at Owens Rd.	$875,000
Dredge Resaca De La Guerra from Owens Rd. to Outfall	$5,375,500
OPTION C.3 TOTALS:	**$6,250,500**
North Main Drain Improvement Totals:	**$45,117,190**

Table 6-30. Estimated Construction Costs of Proposed CCDD1 Flood Mitigation Projects

CCDD1 Proposed Improvements	Costs
OPTION A	
Implement Runoff Controls for New Developments	
Remove and Replace Weir Structure @ Paredes Line Road	$179,400
Detention Pond upstream of UP Railroad - Ditch No. 1	$5,620,500
Detention Pond upstream of UP Railroad - Nopalitos Drain	$5,620,500
Install side weir at Exst. Super Walmart Detention Pond	$229,500
Install side weir at Exst. UP Railroad Detention Pond on Nopalitos Drain	$148,500
Brownsville Golf Center Detention Pond - 180 Acre Feet	$5,882,250
Construct Dana Road Detention Ponds (2)	$11,131,500
Construct Towne North Detention Pond	$7,863,000
Complete CCDD1 Detention Pond	$1,596,000
Construct Robindale Road Detention Pond	$7,995,300
Construct FM 802 Detention Ponds (2)	$23,125,500
Construct Public Works Detention Ponds (2)	$2,975,250
Construct Detention Pond on Minnesota and Austin Road	$4,113,600
Construct Detention Pond on Minnesota near Airport	$5,297,500
Replace FM 3248 (Alton Gloor) culvert with bridge structure	$942,540
Replace Paredes Line Road culvert with Bridge structure	$928,740
Replace Dana Road culvert with Bridge structure	$396,750
Replace Old Port Isabel Road culvert with Bridge structure	$403,650
Replace FM 802 culvert with Bridge structure	$942,540
Replace International Blvd. culvert with Bridge structure	$1,106,760
Replace FM 802 culvert with Bridge structure	$2,452,950
OPTION A TOTALS:	**$88,952,230**
OPTION B	
Line ditch to top of bank from Paredes Line Road to FM 802	$18,232,500
OPTION B TOTALS:	**$18,232,500**
OPTION C	
Line ditch to top of bank from Fm 802 to Outfall	$23,595,000
OPTION C TOTALS:	**$23,595,000**
CCDD No. 1 Ditch No. 1 Improvement Totals:	**$130,779,730**

Table 6-31. Estimated Construction Costs of Proposed RDLG Flood Mitigation Projects

Resaca de la Guerra Proposed Improvements	Costs
Option A	
Quail Hollow Diversion Pump & Detention Pond	$5,347,500
Property Buyouts	$600,000
Option B	
Expand Quail Hollow Diversion Pump & Detention Pond	$5,347,500
Dredge through Quail Hollow	$8,989,499
RESACA DE LA GUERRA IMPROVEMENT TOTALS:	**$20,284,499**

Table 6-32. Estimated Construction Costs of Proposed TR Flood Mitigation Projects

Town Resaca Proposed Improvements	Costs
Impala Pump Station Upgrade	$828,000
Dredge Town Resaca near Brownsville Zoo	$5,616,000
Line Ditch from South WWTP to Impala Pump Station	$1,933,750
Property Buyouts	$750,000
TOWN RESACA IMPROVEMENT TOTALS:	**$9,127,750**

6.6.2 Net Benefits

In order to formulate a plan that is both cost-effective and efficient in reducing flood damages the benefits experienced from implementing a given plan must be quantified and compared against the total investment of applying the plan. The benefits of a given option were calculated by determining the Net Present Value (NPV) of the reduction in expected annual flood damages over the base case (no action) over a twenty-year planning horizon and a forty-year project life. The benefit-cost comparison for each option may be viewed in Tables 6-33 – 6-37. The benefits exceed the cost for all watersheds except for RDLG where the proposed improvements for Resaca de la Guerra do not justify the resulting benefits.

The most cost-effective options for each watershed are those with the highest benefit/cost (B/C) ratios. Those with B/C ratio's greater than one indicate that the benefits outweigh the costs and those with values less than one, such as RDLG, designate options in which the costs exceed the benefits. Examining Tables 6-33 – 6-37 reveals that for CCDD1, Option A undoubtedly provides the most benefit given its' cost of implementation. While all modeled options for CCDD1 have B/C ratio's greater than one, a closer examination of the numbers reveal that most of the benefits are seen from Option A. For example, the NPV of the total costs for Option A total over $37 million and are accompanied by over $630 million in total benefits. This provides over $590 million in total net benefits for Option A. However, more than doubling the total costs, like is done in option B, only adds an additional $8 million dollars in total benefits.

For NMD Option C-3 provides the largest net benefits for any given option. However this would necessitate a $187 million investment. Instead, roughly a quarter of that amount could be invested in the flood control measures included in option B to provide $60 million in total net benefits. This option provides the largest amount of benefits for the cost of the investment and is thus the most cost-effective option for this watershed.

As previously mentioned, the selected options for RDLG had costs that exceeded the benefits. Therefore, no cost-effective recommendation can be made regarding flood mitigation efforts for this watershed based on the results of this study. However, it is recommended that further effort be put towards finding a cost-effective solution to the flooding issues in RDLG especially in the upstream Quail Hollow subdivision.

Only one option was examined for the TR watershed. This option had a total cost of over $8 million but provided over $120 million in net benefits. This option is thus efficient and cost-effective.

A summary of the recommend plan based on the previous discussion may be viewed in Table 6-38. Overall, the implementation of all recommended plans resulted in a net benefit of over $750 million.

Table 6-33. Cost Benefit Comparison for Option A

Watershed	NPV Total Costs Option A	NPV Total Benefits Option A	Benefit/Cost Ratio Option A	Net Benefits
CCDD1	$37,465,205	$630,688,574	16.83	$593,233,368
NMD	$13,718,091	$22,603,664	1.65	$8,885,573
RDLG	$16,946,809	-$1,329,288	-0.08	-
TR	$8,273,771	$120,546,448	14.57	$112,272,676

Table 6-34. Cost Benefit Comparison for Option B

Watershed	NPV Total Costs Option B	NPV Total Benefits Option B	Benefit/Cost Ratio Option B	Net Benefits
CCDD1	$82,648,552	$638,814,534	7.73	$556,165,982
NMD	$47,175,612	$106,048,011	2.25	$58,872,399
RDLG	$31,105,867	$19,116,279	0.61	-

Table 6-35. Cost Benefit Comparison for Option C

Watershed	NPV Total Costs Option C	NPV Total Benefits Option C	Benefit/Cost Ratio Option C	Net Benefits
CCDD1	$137,820,083	$659,007,858	4.78	$521,187,775
NMD	$92,818,969	$116,237,586	1.25	$23,418,617

Table 6-36. Cost Benefit Comparison for Option C-2

Watershed	NPV Total Costs Option C-2	NPV Total Benefits Option C-2	Benefit/Cost Ratio Option C-2	Net Benefits
NMD	$139,513,986	$202,524,200	1.45	$63,010,213

Table 6-37. Cost Benefit Comparison for Option C-3

Watershed	NPV Total Costs Option C-3	NPV Total Benefits Option C-3	Benefit/Cost Ratio Option C-3	Net Benefits
NMD	$187,370,143	$264,848,872	1.41	$77,478,729

Table 6-38. Cost Benefit Comparison for Recommended Flood Damage Reduction Plan

Watershed	NPV Total Costs	NPV Total Benefits	Benefit/Cost Ratio	Net Benefits
CCDD1	$37,465,205	$630,688,574	16.83	$593,223,368
NMD	$47,175,612	$106,048,011	2.25	$58,872,399
RDLG	-	-	-	-
TR	$8,273,771	$120,546,448	14.57	$112,272,676
Total	$92,914,589	$857,283,032	9.23	$764,368,444

6.6.3 Additional Planning Recommendations

In addition to the proposed capital improvements discussed in Section 6.6, there are a number of additional planning needs that are critical to the successful, long term, implementation of the proposed plan. These improvements include realignment of administrative functions, improved data collection and modeling efforts, and are outlined below.

Creation of a Single Regional Drainage Authority – Drainage policy within the study area is overseen by a number of disparate organizations including the City of Brownsville, Cameron County Drainage District No.1, Brownsville Public Utilities Board, and the Brownsville Irrigation District among others. These overlapping jurisdictions diffuse authority and accountability, and make it difficult to develop and implement consistent and cost effective drainage strategies and policies with the study area. The creation of a single regional drainage authority with regulatory and taxing powers is a critical step in the successful implementation of the proposed plan. It would refocus authority and accountability at a single point; would facilitate consistency in the development and implementation of policies; and it would facilitate the development and implementation of cost effective strategies by allowing a regional, rather than sub-watershed, focus.

Extend the H&H Models to Include Resaca del Rancho Viejo and Integrate Individual Watershed Models – One of the salient characteristics of the study area is the interconnectedness of the individual watersheds, and the need to focus solutions that extend beyond individual watersheds. The Resaca del Rancho Viejo (RRV) is still largely undeveloped; but Brownsville is growing at an accelerated rate in its direction, and the

RRV may in addition provide other drainage options for the study area. It is imperative that the hydrologic and hydraulic models developed for Town Resaca, NMD, CCDD1 and Resaca de la Guerra watersheds be extended to cover Resaca del Rancho Viejo so that it can be used to develop flood control strategies now before the watershed develops more fully and solutions become more limited and expensive. At the same time, it is clear that diversions between watersheds, as is the case in the North Main Drain, can provide a set of possible solutions to the drainage problems in the City. Modeling the impact of these diversions at this point is difficult because the current watershed models are not interconnected. Integration of these models, to permit the evaluation of the cross-impacts of diversion, will be a key element in the preliminary engineering phase of the plan.

Install Streamflow and Rainfall Gaging Network – Streamflow data is critical to the development of representative hydraulic and hydrologic models. Unfortunately, there is not a single flow gaging station currently in place within the study area. And although the radar station at the Brownsville airport provides detailed radar rainfall estimates, additional rainfall calibration stations are needed for better rainfall estimates. As the plan development proceeds from its current conceptual level to the design phase, a more accurate representation of the hydraulic and hydrologic behavior of the watersheds in the study area will be required. It is recommended that a streamflow/rainfall gaging plan be developed to guide the number and location of streamflow/rainfall gaging stations throughout the study area as well as in the Resaca del Rancho Viejo.

Training of Brownsville City Personnel – The transfer of the H & H models to City staff is an essential part of the successful implementation of the proposed flood protection plan. Upon completion of the final public hearing and submittal of this report the H & H models for each of the four watersheds will be handed over to City personnel. A training session will be held for select staff on how to run and modify the models as well as on how to analyze the model output.

6.7 Financing Options

This section presents a brief summary of potential options that are available to finance the proposed improvements. Included among the available financing options are Bonds, property taxes, 4B funds, CDBG funds, Corps of Engineers funds, FEMA funds, TWDB funds, development impact fees, and storm water utility fees. The amount, timing and mix of the financing options will need to be developed during the implementation phase.

Bonds – Bond monies are a common vehicle for financing capital improvement projects. The City of Brownsville currently relies on property and sales tax based bond monies to finance street and drainage improvement projects. The City of Brownsville Capital Improvement Plans (CIP) implemented during the past three decades have mainly been used to fund the reconstruction of city streets. The bonding capacity of the City, together with priority and magnitude of competing needs, will determine the amount and timing of the bonds that can be issued to support the proposed projects.

Property Taxes – Cameron County Drainage District No.1 currently collects property tax revenue to fund its operations. To date, the revenues have been used to fund needed maintenance operations and some capital improvements.

4B Funds - In addition to bond funds, the City of Brownsville also has access to 4b sales tax revenues, which may be used to supplement the construction of drainage projects. The key to the use of these funds is the design of detention ponds that can provide not only drainage benefits but can also serve as recreational park areas.

CDBG Grants – Annual Community Development Block Grant fund allocations can be, and have been used in the past, by the City to finance the construction of drainage projects. However, since these funds are limited to construction projects in areas of low to moderate-income families, and there is a significant competition for the limited funds, these grants are best suited to smaller, secondary drainage problems.

TWDB Funds – The Texas Water Development Board (TWDB) provides funds for flood control planning projects. This project, in particular, was funded though the TWDB. Specific projects suited to this funding vehicle include the development of H&H models for Resaca del Rancho Viejo, integration of the study area models, and formulation of technically based development/runoff control policies.

US Corps of Engineers - The COE provides funding for large scale, long term, flood reduction/prevention projects. However, since the required planning effort for obtaining funding approval can be very long (on the order of 10 to 25 years), COE funding is best suited for projects without a short-term priority, such as the dredging of resacas. The City of Brownsville is currently in the planning phase of a large scale Resaca Restoration Project funded by the US Corps of Engineers. This project is evaluating the feasibility of dredging the resacas for environmental enhancement. This project is currently in the feasibility stage.

FEMA – The Federal Emergency Management Agency provides short-term access to funds for the buyout of flood prone areas where flood control projects are not economically feasible.

Runoff Controls and Impact Fees - Impact fees are an alternative option for financing infrastructure capital improvements. It can be used, in conjunction with runoff control policies, to finance the design and construction of regional detention and flood control facilities to compensate for the increased storm water runoff from new developments. The development of a consistent set of technically based runoff control/impact fee policies across the study area is a critical element in the proposed flood control strategy, and can provide an additional source of funds for capital and O&M expenses.

Storm Water Utility Fee –Storm Water Utility Fees can provide an alternative source of revenue to fund capital and O&M costs for drainage improvements. A storm water utility fee could be used to fund not only the drainage improvements, but also the impending requirements imposed on the City by the state-mandated storm water permit program. The City already imposes a fee, collected by BPUB, to cover the expenses of unfunded environmental mandates, which can be modified to help fund drainage improvements.

6.8 CIP Plan Implementation

The number, sequence and timing of the implementation of the proposed CIP projects will depend on the City's financial capacity and whether the benefits of the proposed projects justify the associated costs.

Since the over $130 million price tag of the proposed plan would present a significant financial burden for the City unless it was phased over an extended period of time, a 20-year implementation plan is proposed that consists of four phases. Each phase consists of a 5-year period CIP plan requiring capital investments totaling between $25 and $40 million. The proposed CIP is not fixed in stone. It is a flexible strategy that can be adapted, through the use of the H&H models that were developed as part of this project, to accommodate future changing development patterns, regulatory scenarios, funding options and financial priorities. A summary breakdown of the timing of the proposed capital investments and their associated net present value for each watershed, based on an interest rate of 6%, is presented in Table 6-28.

Table 6-39. Phasing of Proposed Capital Improvement Plan

Watershed	Phase I Capital Costs	Phase II Capital Costs	Phase III Capital Costs	Phase IV Capital Costs	Total Capital Costs	NPV Capital Costs
NMD	$13,890,490	$19,987,500			$33,877,990	$18,917,293
CCDD1	$19,813,650	$19,034,100	$14,014,300	$36,187,470	$89,049,520	$25,442,923
RDLG						
TR	$3,511,750		$5,616,000		$9,127,750	$4,678,092
Total	**$37,215,890**	**$39,021,600**	**$19,630,300**	**$36,187,470**	**$132,055,260**	**$51,191,266**

The proposed CIP phases, including a description of the projects, associated capital costs and potential funding sources, are listed in Tables 6-29 to 6-32, respectively. Potential funding sources are listed as B – Bond Funds, P-Property Taxes, C – CDBG Grants, D – Development Fees, F-FEMA, S- Storm Water Utility Fee, and CO – Corps of Engineers Funds.

Table 6-40. Phase I CIP (Years 1-5)

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Construct Price Road Detention Pond	$855,600	B
Construct Old Port Isabel Rd Detention Pond	$5,791,500	B
Construct City Detention Pond Near Owens Road	$2,035,500	C
Construct Coria Detention Pond	$604,200	B
Construct City Detention Pond Near Airport	$4,036,500	B
Construct levee around southern portion of Airport	$567,190	B
Total NMD:	**$13,890,490**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Funding
Implement Technically Based Runoff Controls for New Developments	---------	S
Remove and Replace Weir Structure @ Paredes Line Road	$179,400	B
Install side weir at Exst. Super Walmart Detention Pond	$229,500	B
Install side weir at Exst. UP Railroad Detention Pond on Nopalitos Drain	$148,500	B
Purchase Land for Dana Road Detention Ponds	$1,402,500	B
Purchase Land for Robindale Road Detention Pond	$874,500	B
Purchase Land for FM 802 Detention Pond	$2,805,000	B
Construct Towne North Detention Pond	$7,863,000	B
Brownsville Golf Center Detention Pond - 180 Acre Feet	$5,882,250	B
Purchase Land for Minnesota & Austin Road Detention Pond	$429,000	B
Total CCDD1:	**$19,813,650**	

Town Resaca		
Proposed Improvement	Estimated Costs	Funding
Property Buyouts	$750,000	F
Impala Pump Station Upgrade	$828,000	B
Line Ditch from South WWTP to Impala Pump Station	$1,933,750	C
Total TR:	**$3,511,750**	
Total Costs:	**$37,215,890**	

Table 6-41. Phase II CIP (Years 6-10)

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Line ditch to top of bank from 77/83 to Airport	$19,987,500	B
Total NMD:	**$19,987,500**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Detention Pond upstream of UP Railroad - Nopalitos Drain	$5,620,500	B
Construct Detention Pond on Minnesota and Austin Road	$3,684,600	B
Construct Dana Road Detention Ponds (2)	$9,729,000	B
Total CCDD1:	**$19,034,100**	
Total Costs:	**$39,021,600**	

Table 6-42. Phase III CIP (Years 11 – 15)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Detention Pond upstream of UP Railroad - Ditch No. 1	$5,620,500	B
Construct Public Works Detention Ponds (2)	$2,975,250	B
Construct Detention Pond on Minnesota near Airport	$5,297,500	B
Complete CCDD1 Detention Pond	$1,596,000	B
Construct Robindale Road Detention Pond	$7,120,800	B
Total CCDD1:	**$22,610,050**	

Town Resaca		
Proposed Improvement	Estimated Costs	Possible Funding Source
Dredge Town Resaca near Brownsville Zoo	$5,616,000	CO
Total TR:	**$5,616,000**	
Total Costs:	**$28,226,050**	

Table 6-43. Phase IV CIP (Years 16 – 20)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Construct FM 802 Detention Ponds (2)	$20,320,500	B
Replace FM 3248 (Alton Gloor) culvert with bridge structure	$942,540	B
Replace Paredes Line Rd. culvert with bridge structure	$928,740	B
Replace Dana Road culvert with bridge structure	$942,540	B
Replace Old Port Isabel Rd. culvert with bridge structure	$403,650	B
Replace FM 802 culvert with bridge structure	$942,540	B
Replace International Blvd. culvert with bridge structure	$1,106,760	B
Replace RR culvert near 48 with bridge structure	$2,004,450	B
Total Costs:	**$27,591,720**	

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TWDB Comments

1. Report does not document extent to which contractor field-verified existing hydrological and land use per Task 2.

   pg. 10, paragraph 2; pg. 13, paragraph 1; pg. 17, section 2.3, paragraph 1

2. Report does not clearly document ranking criteria or how these were developed (with City staff) or applied based on comparisons of present values of expected net benefits of alternative projects per Task 4.2 and Task 7.4. Only final selected projects are presented without showing any comparisons to alternative projects that were not selected based on criteria.

   Pg. 179-182, section 6.6.2

3. Report does not clearly document how City assisted in selecting parameters in Task 4.1

   pg. 17, paragraph 2; pg. 22, paragraph 1; pg. 59, paragraph 2

4. Task 8 of the project scope of work details that City personnel will be trained in the use of flood management model developed by study. I could not locate a discussion in the draft report for the model being transferred to the City of how staff would be trained by the consultant in the use of the model or the proper interpretation of the model results.

   pg. 179, paragraph 3

5. Several items from Task 9.1 (e.g. O&M costs, implementation schedules) are not included in report

   O & M costs are addressed on pg. 186; Implementation schedules were developed in terms of the individual CIPs on pg. 183-185 and discussed on pg. 181-186.

6. Executive Summary, page iii. Typo: “$9 million for RDLG” should be for TR

   Typo corrected

7. Fig. 6-1. DP#5 seems totally out of the watershed and no hydraulic connection between it and the watershed. Explain why it helps reduce flooding.

   Section 6.2.1, pg. 105;

8. Since both RDLG and TR drain into NMD, explain how the improvement options will affect each of the other watersheds if they are all in place.

   Cannot determine at this time. Future efforts include combining individual watersheds into one model. See pg. 177-178 - Extend the H&H Models to Include Resaca del Rancho Viejo and Integrate Individual Watershed Models

9. Appendices need to be listed in Table of Contents

   Appendices added to Table of Contents