Flood Protection Plan - Phase II
Ambiotec Civil Engineering Group, Inc.
August 2011

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Section 5

1004831054_Final Report

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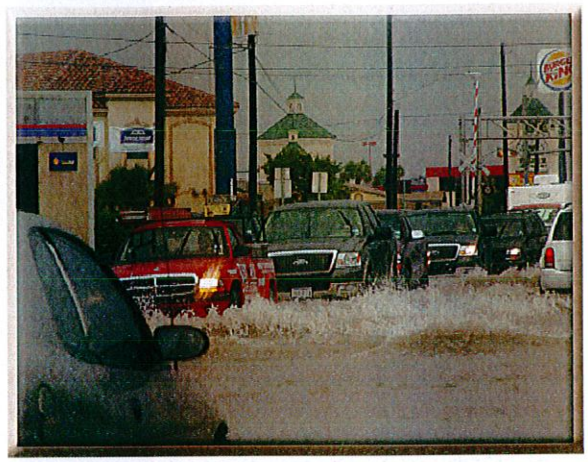

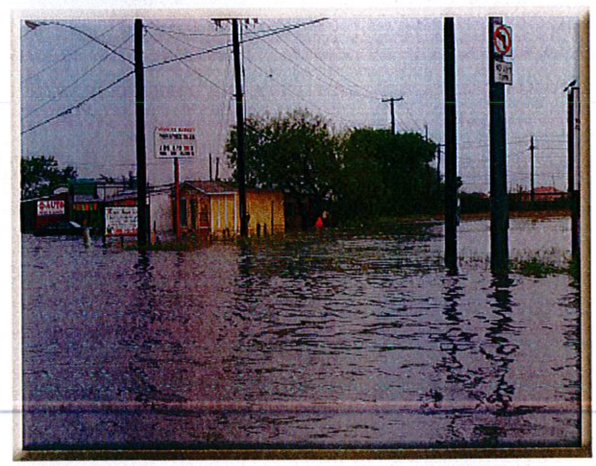

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Ambiotec wishes to acknowledge various individuals and organizations for their valuable contributions to the preparation of the Flood Protection Plan for the City of Brownsville, including Mr. Gilbert Ward of the TWDB and Mr. Ben Medina of the City of Brownsville, as well as Messrs. Carlos Ayala (CCDD1), Carlos Lastra, Doroteo Garcia, Santana Torres (City of Brownsville), Mr. Joe Barrera (Brownsville Irrigation District), Mr. John Bruciak, Ms. Genoveva Gomez, Mr. Jaime Estrada, Mr. David Abrego (Brownsville PUB), and Mr. Ernesto Hinojosa (Cameron County). These individuals without exception shared generously their time and invaluable knowledge and experience.

Executive Summary

The purpose of this study was to expand on the Phase I City of Brownsville Flood Study that was developed in 2006 to expand the study area to include two additional Brownsville watersheds in the Northwest quadrant of the City and to create a linked watershed model to allow for the investigation of interwatershed transfers of stormwater flow between the watershed studied in 2006. This study analyzes several of the gaps identified in the 2006 Flood Study by 1) developing baseline models to assess existing flooding risks in the Resaca del Rancho Viejo (RRV) and Cameron County Drainage District No. 1 Ditch No. 3 (CCDD3) watersheds; 2) examining the impact of storm surge on each watershed; and 3) linking the hydraulic models for those watershed that are hydraulically linked. This study, like the 2006 study, is meant for planning use only and should not be used for engineering design purposes.

The proposed plan is an update of the 2006 Capital Improvement Plan (CIP) that laid out a series of 5 year CIPs to be implemented over a 20-yr period. The plan included both structural and non-structural options that were 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 the 2006 study was 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. The recommended structural options mainly included the construction of detention ponds, especially multi-use detention ponds. 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 were recommended. This update of the 2006 CIP re-examines projected costs, continued feasibility of projects (typically based on land availability) and completion of projects since the 2006 study combined with the investigation of two additional flood mitigation options to arrive at an updated CIP for the City of Brownsville.

The planning area for this study encompasses approximately 109 square miles and includes seven watersheds: Resaca del Rancho Viejo – split into two separate watersheds (Upper Resaca del Rancho Viejo (URRV) and Lower Resaca del Rancho Viejo (LRRV)), Cameron County Drainage District No. 1 Ditch No. 3 (CCDD3), Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1), Resaca de la Guerra (RDLG), North Main Drain (NMD), and Town Resaca (TR). A discussion of the models developed for the expanded planning area (URRV, LRRV, and CCDD3), the two additional flood mitigation alternatives that were analyzed in this phase II study and a brief discussion of the overall CIP update are described in subsequent paragraphs as well as in the body of the report. An analysis of the existing conditions in the expanded planning area revealed varied levels of flooding throughout the area with the largest areas in CCDD3. In the URRV/CCDD3 watersheds (which were linked) nearly 30% of the entire land area is inundated with water for the 100-yr storm event. In the LRRV watershed, approximately 18% of the land area was inundated with water during a 100-yr event. The 100-yr floodplains for both watershed areas may be viewed in Figures ES-1 and ES-2.

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

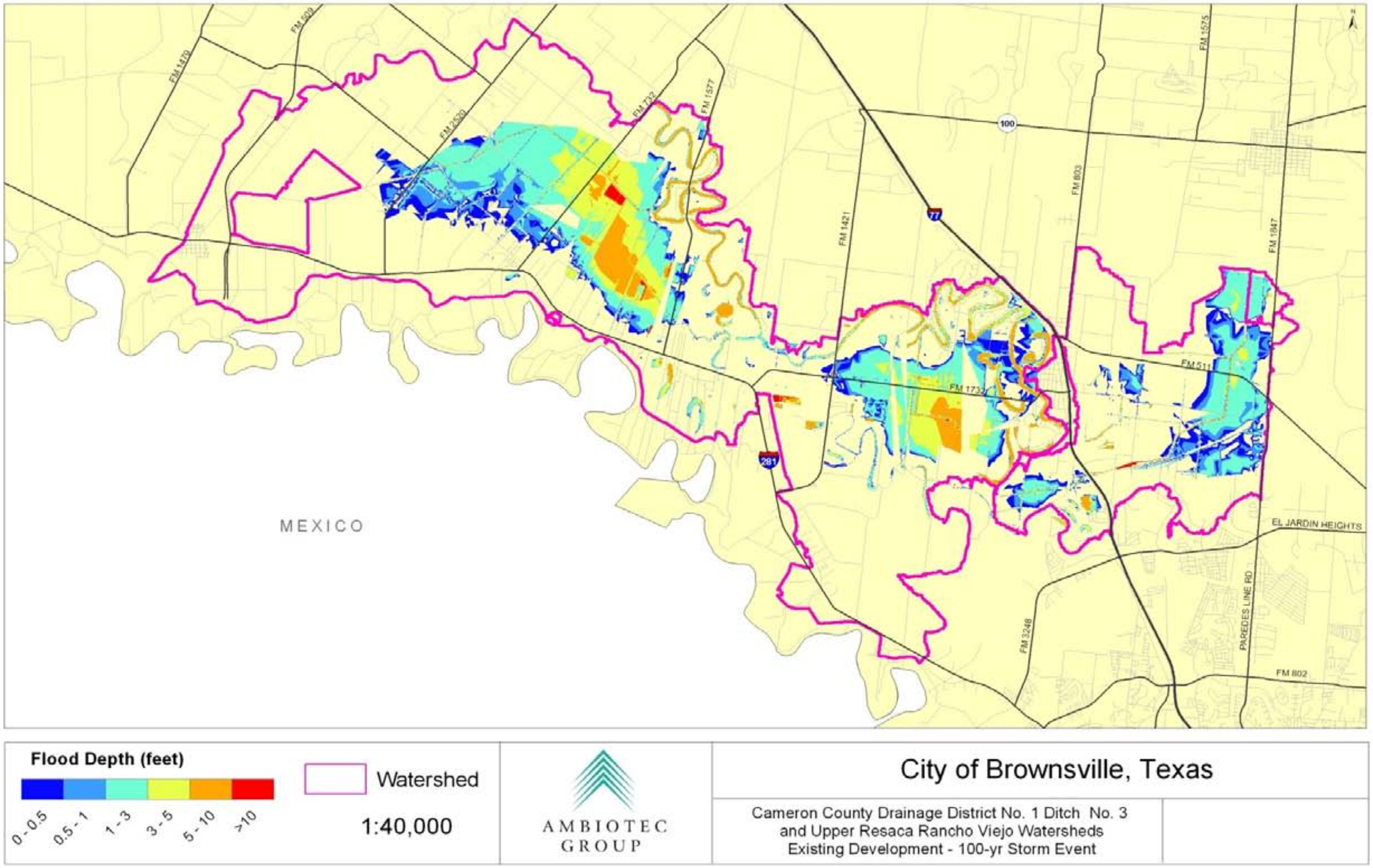


Figure ES-1. Existing development floodplain for the URRV/CCDD3 watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

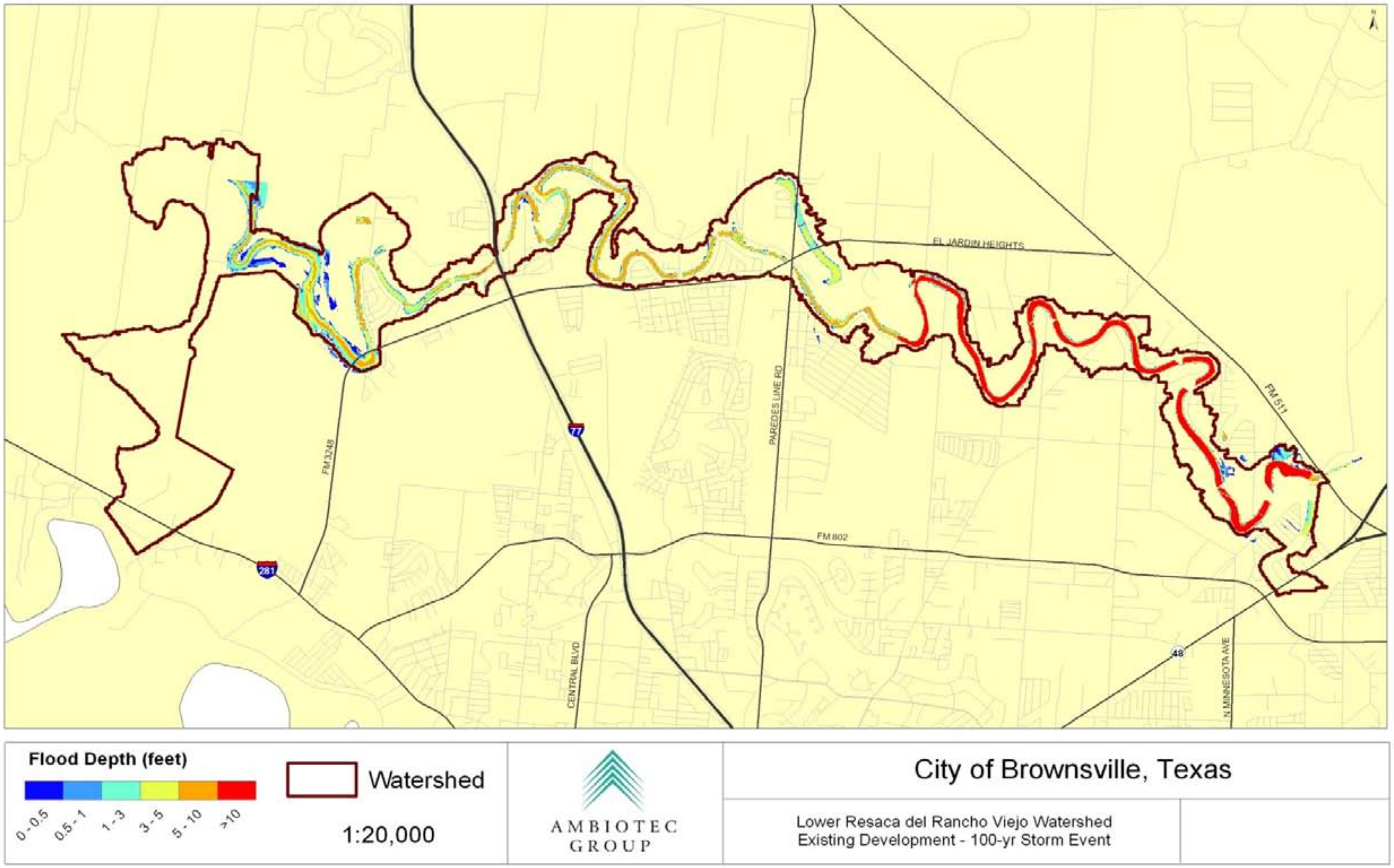


Figure ES-2. Existing development floodplain for the LRRV watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area fully developing the study area without runoff controls would have on flow rates and floodplain depths and areas in the drainage ditches and resacas. In URRV/CCDD3 the amount of inundated area expanded to over 30% of the entire land area and in LRRV it expanded to nearly 20% of the land area revealing a very modest increase in overall floodplain area. The 100-yr floodplains for both watershed areas under the full development scenario may be viewed in figure ES-3 and ES-4.

To link the hydraulic models developed during the 2006 study it was necessary to first re-develop the hydrologic models for the Resaca de la Guerra (RDLG) and Town Resaca (TR) watersheds which were previously developed using a different type of modeling software in HEC-HMS format using the Clark Method for the sake of consistency with the rest of the planning area. While the converted resaca models used the same watershed delineation and much of the same data used to develop the 2006 models, TC&R values were developed using updated landuse and percent impervious values.

A single, linked HEC-RAS (hydraulic model) was developed for those three watersheds that share hydraulic connections: RDLG, NMD, and TR. While many of the same data inputs from the 2006 study were used to develop the merged model (cross-sections and topographic data), more up-to-date landuse and impervious data as well as updated culvert data (where culverts were replaced and data was available) was incorporated into the linked model. The result of this revealed that over 85% of the combined watershed is now developed with varying levels of residential, commercial and industrial landuse. The linked model has the advantage over the models utilized in 2006 of better representing the backwater effects that result from varying water surface elevations in the three systems in instances where the models are hydraulically connected. The results of this analysis revealed that the 100-yr floodplain would inundate approximately 33% of the entire watershed area which is a 6.6% increase in area over that of the 2006 analysis. The 100-yr floodplain for the updated, merged model may be viewed in figure ES-5.

The storm surge analysis that was completed in this Phase II report attempted to analyze the impact that a Hurricane Katrina magnitude storm would have on the Brownsville region with respect to the ability of the City's drainage features to drain. In other words, this analysis did not identify every portion of the City that would be subjected to flood waters from an extreme storm surge but instead examined the backwater effect that such a surge would cause on local drainage systems. This analysis was completed using the maximum storm surge elevation that was observed during Hurricane Katrina of 28-ft. It should be noted that Hurricane Katrina was an extreme and rare event and is not likely from a probability standpoint to be witnesses again for many years. However, this analysis was meant to illustrate a "worst case" scenario. The analysis of a 100-yr rainfall event combined with a 28-ft storm surge resulted in a floodplain inundation area of nearly 35% of the URRV/CCDD3 land area, 47% of LRRV, 70% of CCDD1, and 45% for the merged RDLG, NMD, and TR land area. The resulting floodplains may be viewed in figures ES-6 –ES-9.

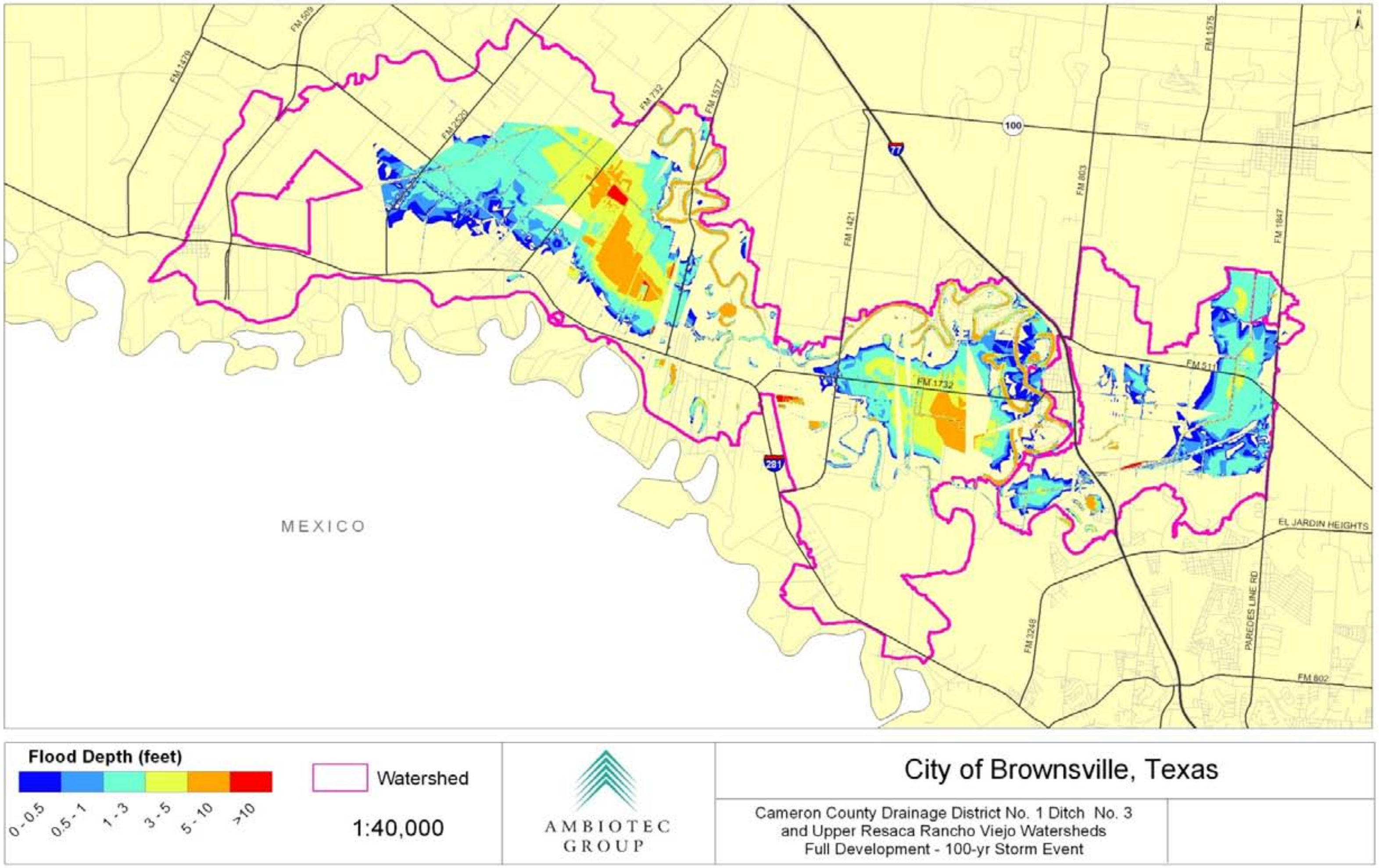
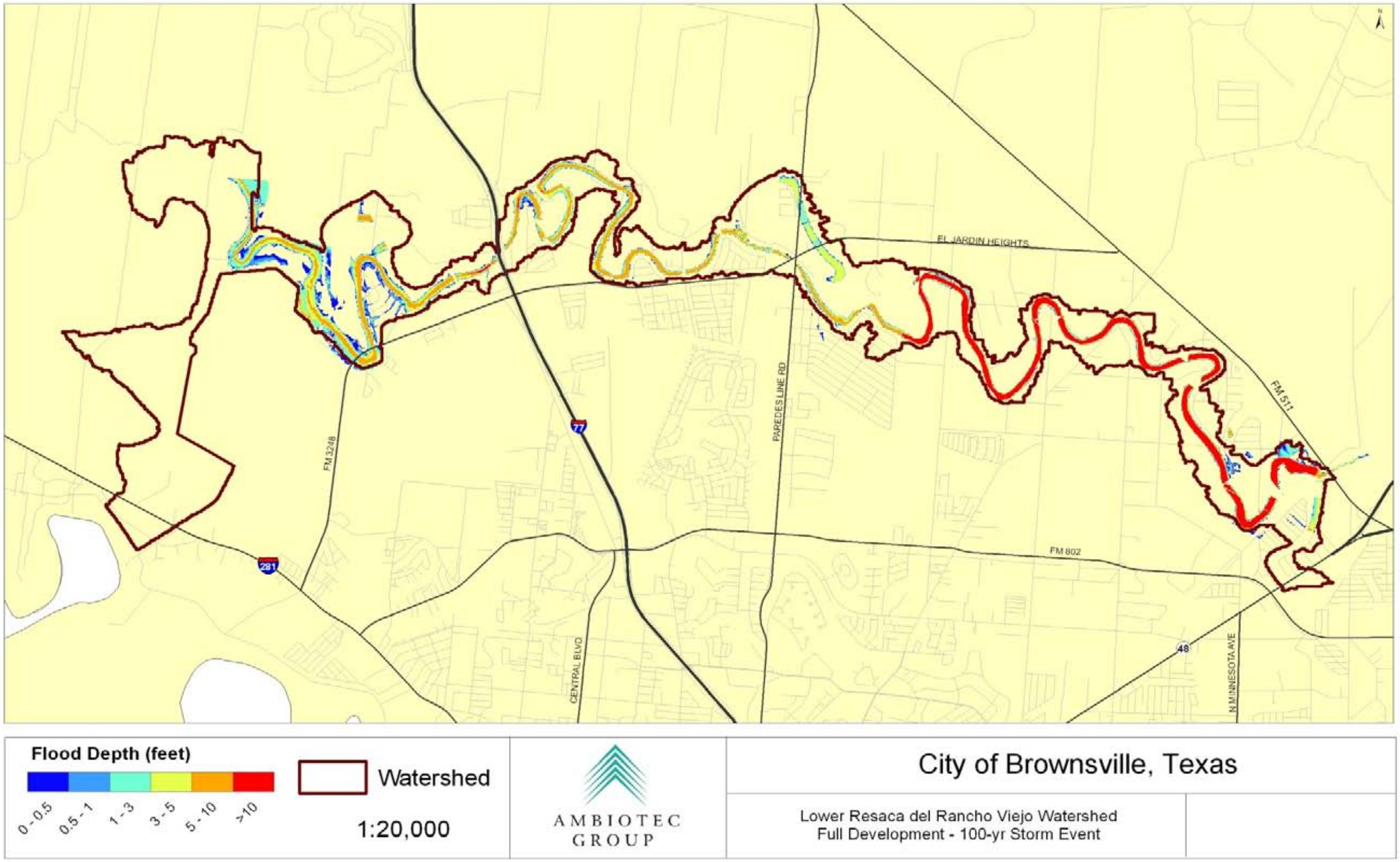


Figure ES-3. Full development floodplain for the URRV/CCDD3 watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area



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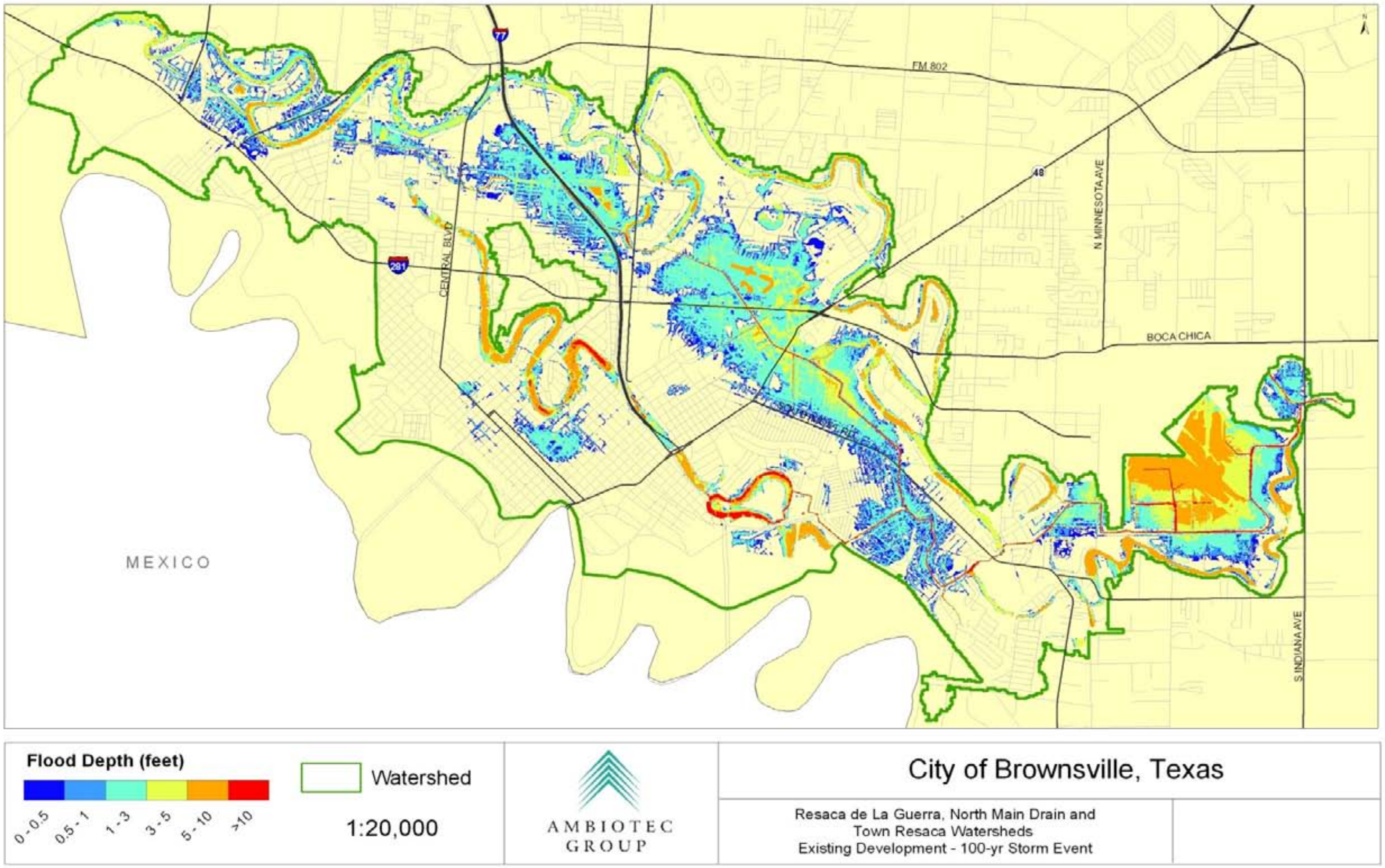

Figure ES-5. Existing development floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

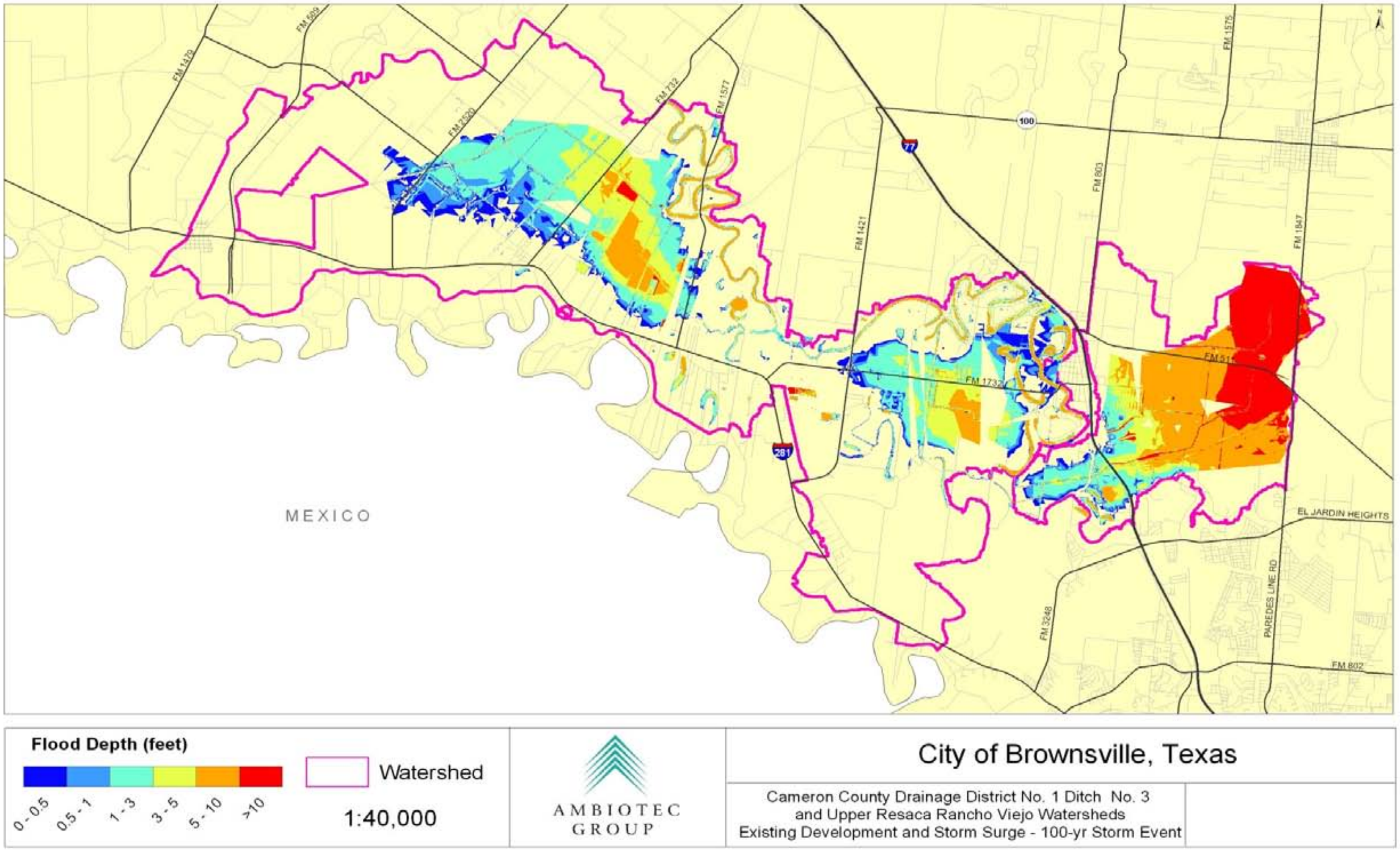

Figure ES-6. Floodplain for the URRV/CCDD3 watersheds under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

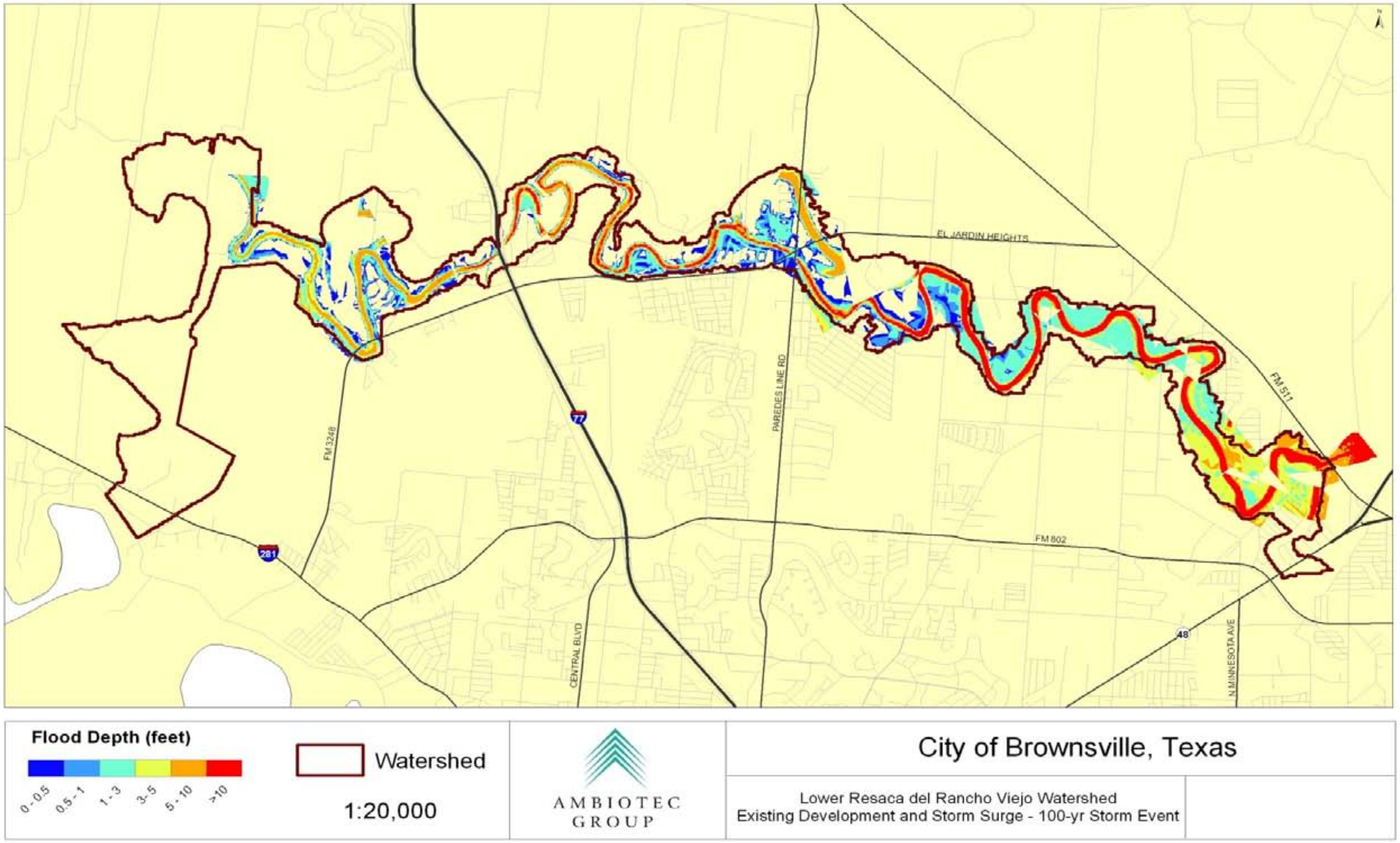

Figure ES-7. Floodplain for the LRRV watersheds under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

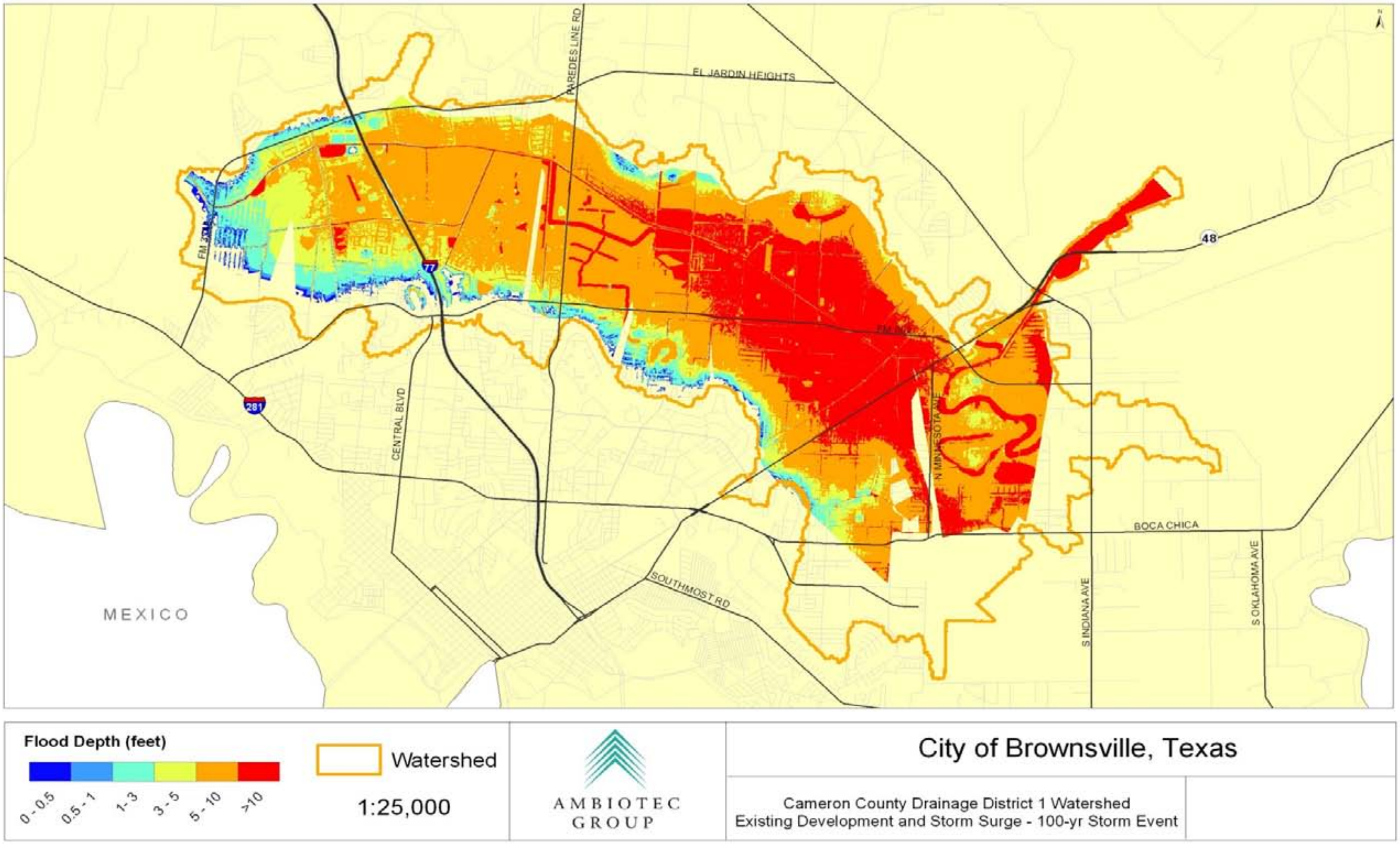

Figure ES-8. Floodplain for the CCDD1 watershed under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

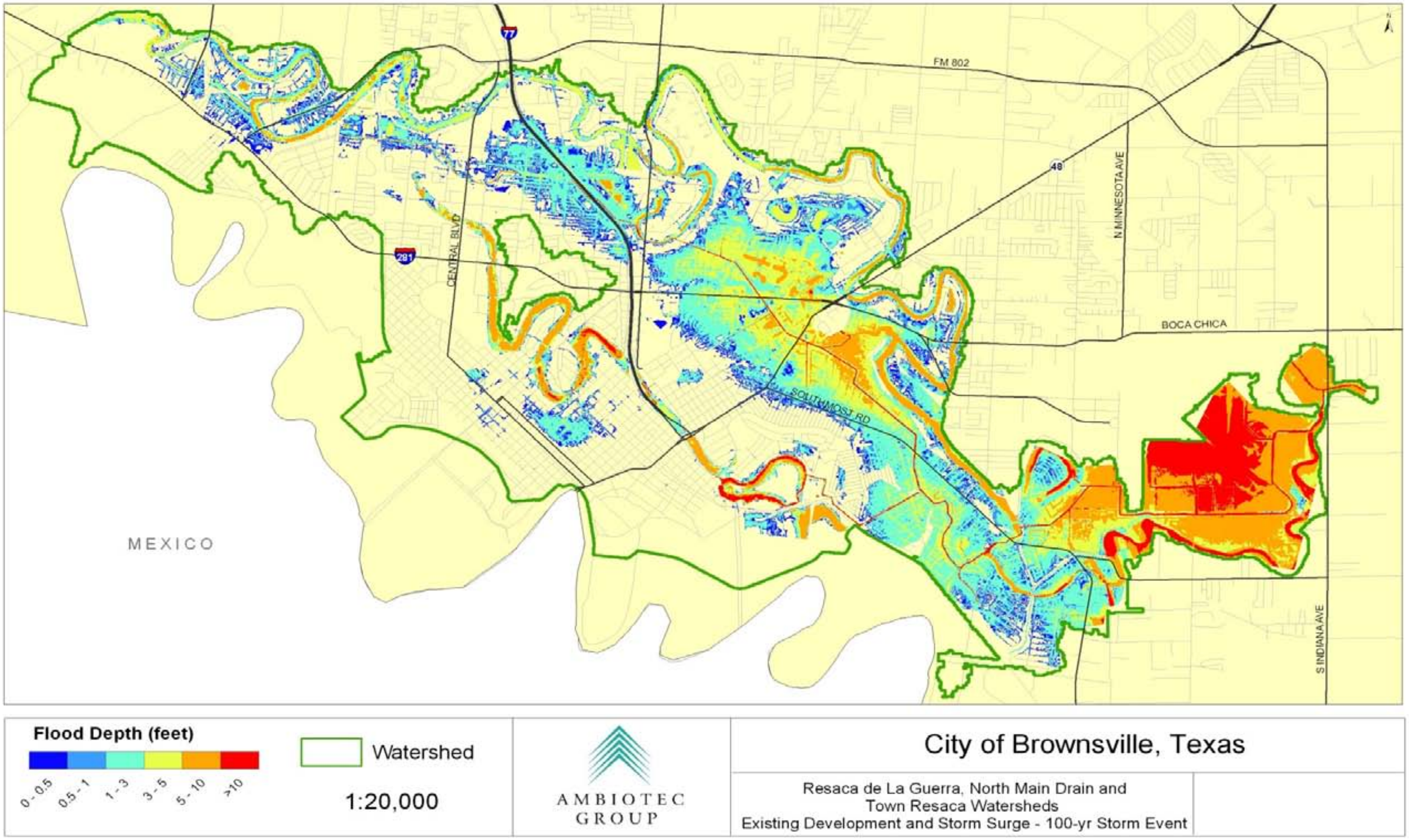

Figure ES-9. Floodplain for the merged model (RDLG, NMD, and TR watersheds) under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

The Flood Protection Plan from 2006 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. This Phase II analysis updated the 2006 CIP to include additional flood mitigation options, remove projects that have already been completed or are no longer feasible and provide updated cost estimates and timeline for completing the rest of the CIP.

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 $145 million and includes approximately $31 million in improvements for the North Main Drain, $102 million for CCDD1, $2.5 million for RDLG and $11 million for Town Resaca. 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 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
NMD	$9,716,700	$21,125,000			$30,841,700
CCDD1	$13,352,950	$24,214,400	$26,373,950	$38,007,000	$101,948,300
RDLG	$2,471,250				$2,471,250
TR	$4,619,000		$6,565,000		$11,184,000
Total	**$30,159,900**	**$45,214,400**	**$32,938,950**	**$38,007,000**	**$146,445,250**

Overall, the proposed plan would result in an investment of approximately $7.3 million/year over a 20-yr period to fully implement all of the proposed projects. Clearly, this will present a significant financial burden to the City and local drainage entities and provides clear evidence of the need for the capture of external funding sources as well as a cooperative approach between local and regional drainage entities to protect the City and southern Cameron County from flood damages.

A list of the proposed improvements along with potential funding sources, grouped by phase, is presented in Tables ES-2 – ES-5. 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. Additional recommendations for the plan implementation include creation of a single regional drainage authority, installation of streamflow and rainfall gages, development of a flood alert system, and continued coordination and cooperation between current regulatory entities.

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

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Construct Price Road Detention Pond	$1,200,600	B
Complete Design and Construction of City Detention Pond Near Owens Road (Currently Under Design)		
Construct City Detention Pond Near Airport	$7,486,500	B
Construct levee around southern portion of Airport	$1,029,600	B
Total NMD:	**$9,716,700**	

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	$365,700	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	$2,125,000	B
Purchase Land for Robindale Road Detention Pond	$1,325,000	B
Purchase Land for FM 802 Detention Pond	$4,250,000	B
Complete Remaining Portion of Towne North Detention Pond	$4,259,250	B
Purchase Land for Minnesota & Austin Road Detention Pond	$650,000	B
Total CCDD1:	**$13,352,950**	

Town Resaca		
Proposed Improvement	Estimated Costs	Funding
Property Buyouts	$950,000	F
Impala Pump Station Upgrade	$1,621,500	B
Line Ditch from South WWTP to Impala Pump Station	$2,047,500	C
Total TR:	**$4,619,000**	

****CIP I continued on following page

Resaca de la Guerra		
Proposed Improvement	Estimated Costs	Funding
Culvert Improvement at 5 VICC Culverts	$675,000	
Culvert Improvement at Upstream Morningside Rd. Crossing	$135,000	
Culvert Improvement at Downstream Morningside Rd. Crossing	$135,000	
Culvert Improvement at Shidler Rd.	$135,000	
Culvert Improvement at Price Rd.	$135,000	
Culvert Improvement at Eagle Drive	$135,000	
Pump Improvement at Outfall of RDLG	$1,121,250	
Total RDLG:	**$2,471,250**	
Total Costs:	**$30,159,900**	

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 RDLG confluence	$16,900,000	B
Line ditch to top of bank from RDLG confluence to Airport	$4,225,000	B
Total NMD:	**$21,125,000**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
		B
Construct Detention Pond on Minnesota and Austin Road	$6,707,400	B
Construct Dana Road Detention Ponds (2)	$17,382,000	B
Total CCDD1:	**$24,089,400**	
Total Costs:	**$45,214,400**	

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

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Replace FM 802 culvert with 3, 10'x10' box culverts	$1,776,750	B
Replace Old Port Isabel Rd. culvert with 3, 10' x 10' boxes	$1,621,500	B
Construct Detention Pond on Minnesota near Airport	$8,321,000	B
Replace Paredes Line Rd. culvert with 3, 10' x 10' boxes	$1,690,500	B
Construct Robindale Road Detention Pond	$12,964,200	B
Total CCDD1:	**$26,373,950**	

Town Resaca		
Proposed Improvement	Estimated Costs	Possible Funding Source
Dredge Town Resaca near Brownsville Zoo	$6,565,000	CO
Total TR:	**$6,565,000**	
Total Costs:	**$32,938,950**	

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)	$36,316,500	B
Replace FM 3248 (Alton Gloor) culvert with 3, 10' x 10' boxes	$1,690,500	B
Total Costs:	**$38,007,000**	

Disclaimer

The following flood protection report is a conceptual level analysis of a seven watershed area in the City of Brownsville (Upper Resaca del Rancho Viejo, Lower Resaca del Rancho Viejo, Cameron County Drainage District No. 1 Ditch No. 3, Cameron County Drainage District No. 1 Ditch No. 1, Resaca de la Guerra, North Main Drain, and Town Resaca) in the City of Brownsville. The purpose of the report is to identify flood-prone areas and draft conceptual based flood control plans at a regional scale. It is not intended for site specific, detailed engineering, design plans.

1.0 Introduction

1.1 Purpose of Study

In 2006 a Flood Protection Plan was developed for the City of Brownsville to address persistent drainage issues throughout the City. The plan was focused on 4 major drainage systems in the City and their corresponding watershed areas. These drainage systems included: Cameron County Drainage District No. 1 Ditch No. 1 (CCDD1), Resaca de la Guerra (RDLG), North Main Drain (NMD), and Town Resaca (TR). While specific drainage issues and causes are described in detail throughout the 2006 report, there was also identified a need for an expansion of the planning area and incorporation of model linkages and storm surge components to the existing planning area (Ambiotec, 2006).

The expansion in study area is needed to establish baseline flooding conditions in the Northwest Quadrant of the City which has been and will continue to be the major area of growth in the future (City of Brownsville, 2007). The Northwest quadrant of the City discussed in the Northwest Brownsville Land Use Study and Plan (2007) is largely drained by the Resaca del Rancho Viejo Resaca (RRV) system and Cameron County Drainage District No. 1 Ditch No. 3 (CCDD3). Similar to the watersheds discussed in the 2006 study, the RRV and CCDD3 watershed areas are characterized by flat slopes and clay-rich soils. This results in low soil permeability, ponding of stormwater and overall conditions that contribute to high probabilities of flooding during rainfall events.

This study serves to fill the gap identified in the 2006 study by developing baseline models to assess existing flooding risks in the RRV and CCDD3 watersheds. Additionally, a storm surge component was incorporated into the hydraulic models and the feasibility of inter-watershed transfers of stormwater flow were examined through a linking of hydraulic models for those areas that are hydraulically linked. This study, like the 2006 study, is meant for planning use only and should not be used for engineering design purposes.

1.2 Description of Planning Area

The planning area for this study encompasses approximately 109 square miles and includes 7 watersheds. The watershed areas included in the study are the Upper Resaca del Rancho Viejo Watershed (URRV), the Lower Resaca del Rancho Viejo Watershed (LRRV), Cameron County Drainage District No. 1 Ditch 3 (CCDD3), Cameron County Drainage District No. 1 Ditch 1 (CCDD1), Resaca de la Guerra (RDLG), North Main Drain (NMD) and Town Resaca (TR) (Figure1-1). The four watersheds that are listed here that were part of the 2006 Study make up 43.6 square mile of the planning area and were included in this study for analysis of storm surge and inter-watershed transfers of flow that were not evaluated previously. The remaining three watersheds (URRV, LRRV, and CCDD3) are new areas that have not previously been studied.

To evaluate inter-watershed transfers of stormwater flow the hydraulic models for RDLG, NMD and TR watersheds from the previous study were merged into a single model as all 3 systems are interconnected. The CCDD1 drainage system was not modified or merged and only included in the planning area to evaluate the impacts of a storm surge. The Resaca del Rancho Viejo (RRV) system

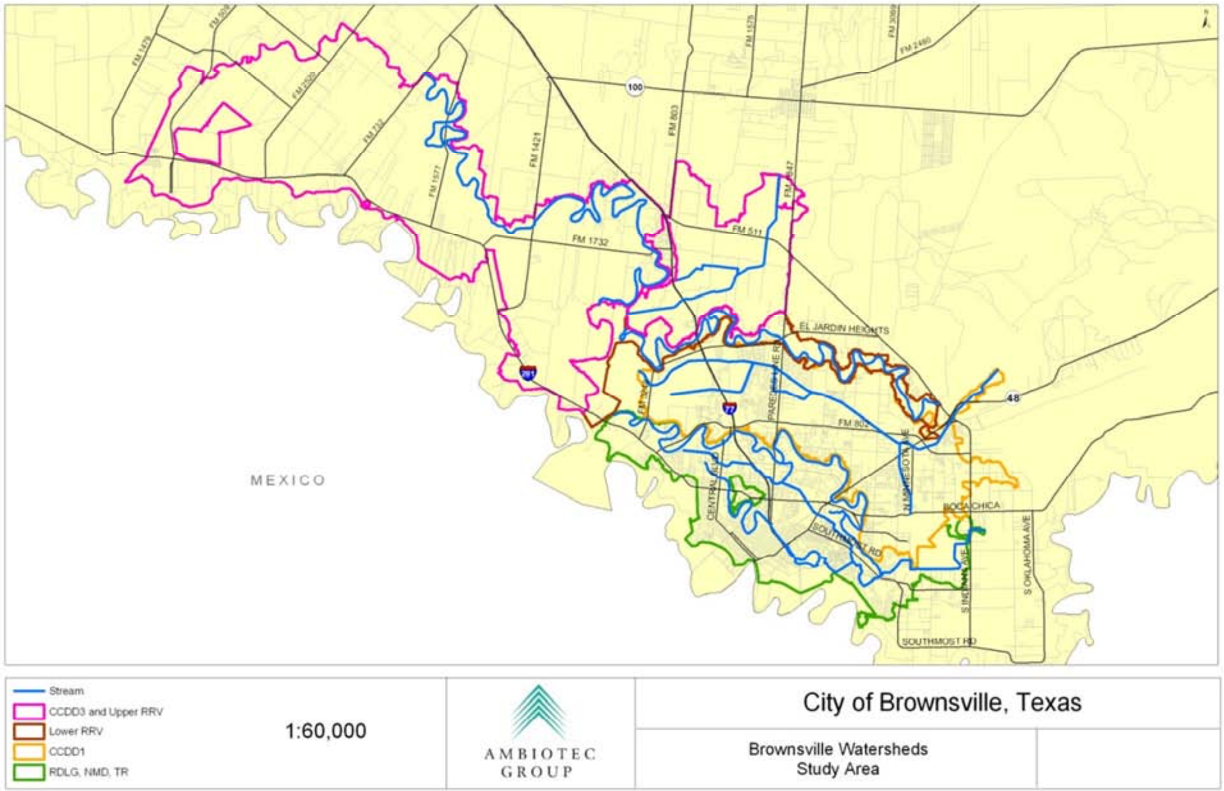

Figure 1-1. Brownsville Flood Study Phase II Planning Area

(Figure 1-2) was split into two watershed areas (upper and lower) due to a drainage structure that routes flow from the upper portion of RRV to CCDD3. As such, while the hydrologic models for each watershed were developed separately to calculate flow, the hydraulic models were merged into one single model similarly to the way that RDLG, NMD, and TR were merged. This allows for a more accurate evaluation of backflow effects between the two systems. The hydraulics of the lower portion of RRV (LRRV) was evaluated independently of the other drainage systems. While a significant portion of the URRV watershed lies outside of the Brownsville City Limits (Figure 1-2) it was necessary to evaluate the entire watershed area to estimate the flow entering the portions of the URRV and CCDD3 drainage systems that are within City Limits.

Typical elevations throughout the study area range from about 50-ft msl in the northwest portion of the study area to less than 5-ft msl near the outfall of CCDD1. All elevation data for the expanded study area was obtained from Light Detection and Ranging (LIDAR) data collected for the International Boundary and Water Commission (IBWC) in 2006. It should be noted that while the data was flown in 2006 the refined product was not available until 2008.

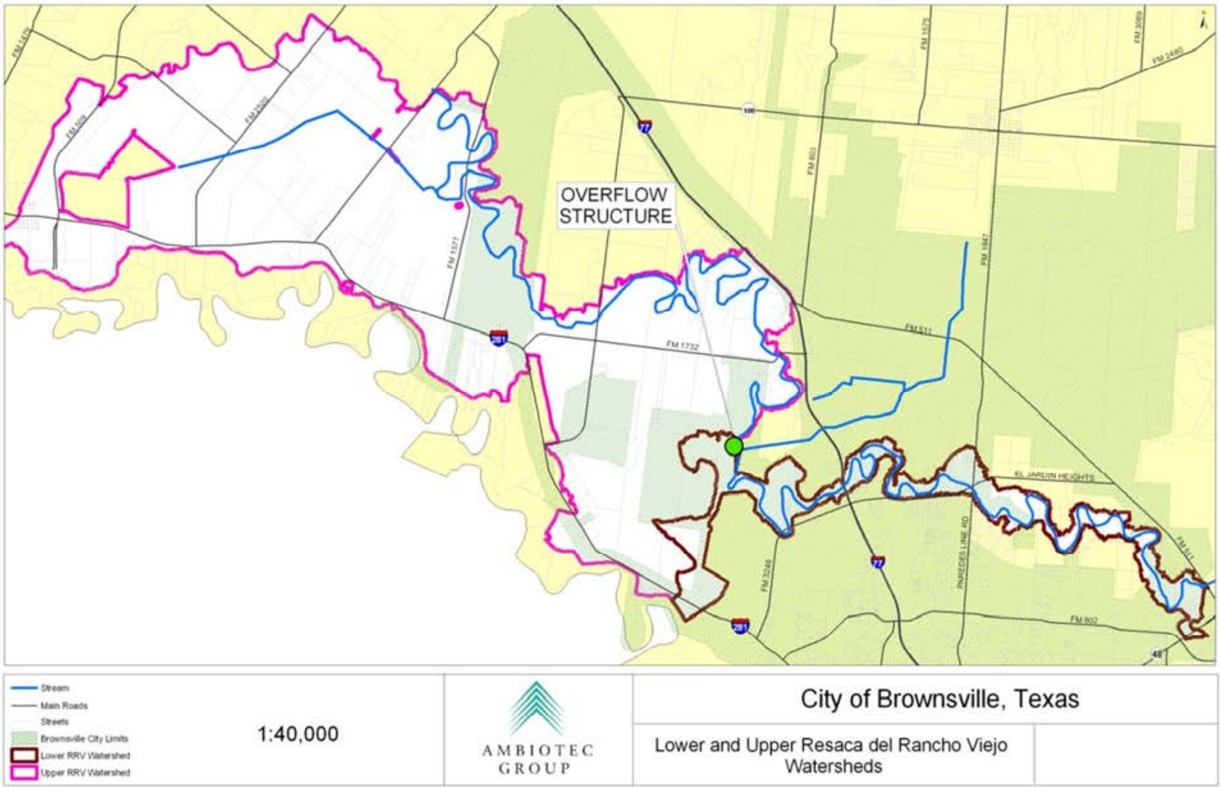

Figure 1-2. The Upper and Lower RRV Resaca watershed boundaries with the overflow structure that splits one drainage feature from the other. The green overlay represents the Brownsville City Limits.

1.3 Project Scope

The specific project tasks that were completed for this project are listed and described below.

1) Compile and Review Existing Hydrologic and Land Use Data for RRV and CCDD3

   Data was collected from previous reports, as well as from City, State, and other governmental agencies. Data collected includes culvert data, Geographic Information Systems (GIS) data describing overland characteristics such as soils, landuse, zoning, etc., and meteorological data. Data was collected for the entire planning area and used for model development and to assess known flood-prone areas.

2) Collect, Field Verify, Compile and Review Additional Information to Update Existing Hydrologic and Land Use Information for Expanded Planning Area

   Survey crews were sent into the field to collect data on over 70 culverts throughout RRV and CCDD3. Data collected included culvert type, size, flowlines and top of structure elevations which was used to develop hydraulic models for the RRV and CCDD3 system. This data was used to update and enhance the existing culvert data set from previous reports, maps and local knowledge. Landuse data was collected in a GIS format from the Multi-

Resolution Land Characteristics Consortium (MRLC) National Land Cover Database and spot checked with aerial photos and field reconnaissance. An examination of the impact on stormwater flow from changes in landuse resulting in an increase of impermeable area along with an examination of projected future growth was conducted and discussed in subsequent sections.

3) Develop New RRV and CCDD3 Flood Management Models Based on Current and Future Conditions

Data collected in Tasks 1 and 2 was used to parameterize planning area watersheds and develop hydrologic and hydraulic models under existing runoff conditions. The models were then run using design frequency rainfall events to reveal probable areas of flooding and predicted water surface elevations.

4) Re-create RDLG and TR models in HEC-HMS so models can be linked

The previous, 2006 flood study used a distributed model called Vflo. It was determined during the previous study that using river reaches with a modified puls storage routing technique, the individual Resaca "pools" could be represented using HEC-HMS software. The major advantage in this is that first of all, the HEC-HMS software is free, tested, and used by many other public and private entities around the country to estimate storm flows. The other major advantage is that HEC-HMS was used to model other Brownsville watersheds so by converting the RDLG and TR models into HEC-HMS format, consistency may be preserved across the entire study area and inter-watershed flow transfers can more accurately be investigated.

5) Link four previously developed models and the newly create model for RRV together and add a storm surge component to the models

The HEC-RAS hydraulic models for the RDLG, NMD, and TR watersheds were merged into one singular model. CCDD1 remained independent because the linkages between RDLG, NMD, and TR are not shared with the CCDD1 watershed and because the outfall for CCDD1 is different than that of the merged RDLG, NMD, and TR system. While all three systems in the merged model discharge to ship channel near the Port of Brownsville, CCDD1 drains independently into San Martin Lake.

The linked model along with the models developed for CCDD1, LRRV, and URRV/CCDD3 were then evaluated assuming a storm surge of 28-ft by placing a boundary condition at the outfall of each hydraulic model. The storm surge of 28-ft was selected because was recorded as the maximum storm surge experienced by coastal Mississippi during Hurricane Katrina in 2005 (Knabb, Rhome, and Brown, 2005). While this is an extreme situation the purpose of this analysis was to establish a "worst case scenario" type of situation.

6) Develop Alternative Structural and Non-Structural Flood Management Strategies including inter-watershed transfers using new linked flood management model and investigating the use of RRV as a detention area.

   The linked models were used to develop and analyze various flood mitigation improvements, including an inter-watershed transfer of flow between NMD and RDLG. The use of RRV as a recipient of additional flow from CCDD1 was eliminated due to existing areas of the RRV Resaca that have experienced repeated incidences of flooding over the last several years, especially in the vicinity of Cameron Park. Due to this factor and several meetings with various local drainage personnel, it was revealed that there would be resistance to transferring additional flow to this system. Each mitigation strategy was analyzed in terms of predicted damage reduction from implementation and overall capital and O&M costs.

7) Compare Alternative Strategies and Select Candidate Cost Effective Options Based on Results of a Cost-Benefit Analysis.

   Alternative strategies were evaluated from a cost-benefit perspective and used to update the flood mitigation plan developed from the previous study.

8) Presentation and Selection of Flood Management Plan

   The results of the analysis were summarized in text, figures and profiles and presented to the City and the public in a final public meeting.

9) Update Final Flood Management Plan Report Including Results for RRV and CCDD3

   In addition to the public meeting, the results of the analysis are also summarized in this report that has been submitted to both the City of Brownsville and the TWDB for review. All final comments will be incorporated into the final version of the report.

10) Progress Meetings and Reports

   Monthly progress reports were prepared and submitted to the City. Additionally, numerous meetings and phone calls took place with local drainage and planning personnel. The information collected during these meetings was used to complete the analysis.

   Three public hearings were conducted throughout the study timeline. The first kick-off meeting was held in June of 2007. The second public meeting was held in June of 2009. The final public hearing was held in March of 2011.

2.0 Data Collection and Description of Modeling Approach and Damage Assessment

2.1 Data Collection

The data used to develop hydrologic and hydraulic models in this study was largely collected within a Geographic Information Systems (GIS) format. A more detailed description of GIS and specific functions within the software is described later in Section 2. The data that was collected came from a variety of sources including: The City of Brownsville, Brownsville Public Utilities Board, various internet sources (subsequently described) and the International Boundary and Water Commission (IBWC). Descriptions of each major dataset along with its source will be discussed in subsequent sections.

2.1.1 Landuse Data

Land use data was obtained from the Multi-Resolution Land Characteristics Consortium (MRLC) National Land Cover Database. The most recently available data was based off of 2006 characterizations. This is the same date that the most recent aerial photographs that are available for the Brownsville region. These aerials were downloaded from the Brownsville Public Utilities Board FTP site and were used along with field investigations for visual verification of landuse data. The land use types and their corresponding classification number may be viewed in Table 2-1. The landuse map used for this analysis is displayed in Figure 2-1. As may be observed in the landuse map, the southeastern portion of the map which is where the City of Brownsville is located, is largely pink to red in color indicating that the area is largely developed.

2.1.2 Elevation Data

Topographic data for this analysis was obtained from the U.S. International Boundary and Water Commission (IBWC). The data was collected for the IBWC in conjunction with the Texas Water Development Board (TWDB) in 2006 along all coastal and border counties. This data was used to develop digital elevation models (DEMs) used within the ArcMap software with the Spatial Analyst, 3D Analyst, HEC-GeoHMS, and HEC-GeoRAS extensions to develop both the HEC-HMS and HEC-RAS models described in subsequent sections. Additionally, cross-section data was collected along with the culvert data described in Section 2.1.3 to adjust invert elevations for the drainage ditches and Resaca systems.

	Grid Code	Description
Water	11	Open Water
	12	Perennial Ice, Snow
Developed	21	Developed Open Space
	22	Developed Low Intensity
	23	Developed Medium Intensity
	24	Developed High Intensity
Barren	30	Bare Rock, Sand/Clay
Forested Upland	41	Deciduous Forest
	42	Evergreen Forest
	43	Mixed forest
Shrubland	52	Shrubland
Herbaceous Upland	71	Grasslands, Herbaceous
Herbaceous Planted, Cultivated	81	Pasture, Hay
	82	Row Crops
Wetlands	90	Woody Wetlands
	95	Emergent Herbaceous Wetlands

Table 2-1. Land Use Classification

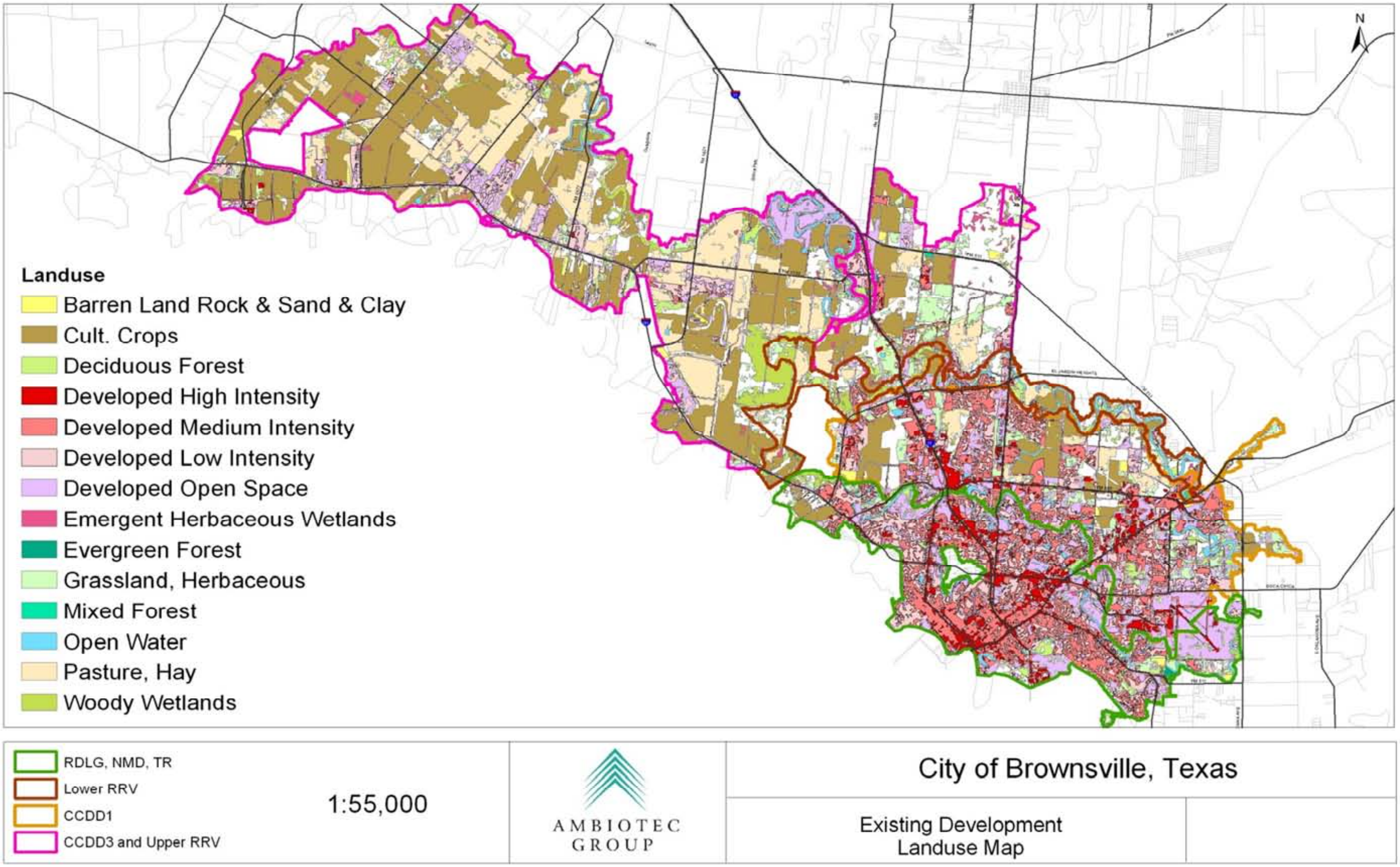

Figure 2-1. Existing Development Landuse map based on 2006 National Land Cover Database

2.1.3 Culvert Data Collection

Culvert data for the expanded planning area (URRV, LRRV, and CCDD3 Watersheds) was collected through an extensive field collection effort and augmented with previous reports, design plans, and coordination with local drainage entities. In the three-watershed study expansion area over 50 culverts were identified and visited and information was collected for incorporation into the hydraulic models. The general information of 47 of these culverts is listed in Tables in subsequent sections of the report. Additional culvert data was collected but later removed from the planning area as watershed boundary delineations were finalized. For the merged model that included three of the watersheds (RDLG, NMD, and TR) from the 2006 study, culvert data from the previous report was used except in cases where known culvert improvements were made and information could be provided by the City.

2.1.4 Rainfall Data

The rainfall data used for this study, like the 2006 study, was design storm data from the U.S. NWS TP 40 report (Hershfield, 1961). A design storm is a theoretical precipitation event used as the basis of design for a hydrologic system (Bedient and Huber, 2002). A design storm is defined by the amount of precipitation and its distribution across a given watershed both temporally and spatially. The U.S. NWS TP 40 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 a storm is the probability of a given 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. As discussed in the 2006 study, 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. It is important to note that for the analyses completed during this study, it is assumed that each given rainfall total is experienced across the entire watershed area. In reality, it is somewhat unlikely that the same exact rainfall totals and intensities would be experienced across the entire watershed area. However, because there are an infinite number of possibilities for rainfall distribution, it is a

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

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

necessary assumption for a planning level analysis such as this one. As a result, floodplains that result from actual events with varying levels of rainfall totals and intensities across a given watershed, are likely to change the overall appearance of the floodplain.

2.1.5 Damage Assessment Data

To estimate damages to buildings (homes and businesses) due to flooding from a given frequency rainfall event, two pieces of information were required. First, it was necessary to geographically locate all buildings or structures within the watershed area. Second, reasonable valuations of each building needed to be assigned. In the 2006 study the data collected to complete this analysis came from a point-type GIS shapefile from the Brownsville Public Utilities Board (BPUB) that identified structures within their service area. It was then assumed that each point (representing a structure) had an average valuation of $60,000. One noted problem with the use of this file during the 2006 analysis was that the point file included relatively small structures like gazebos, garages and tool sheds that not only have relatively little value compared to major buildings (i.e. homes and businesses) but whose values are likely included with the valuation of the primary building on a site. However, because this was the best data available at the time and it was the relative difference between damages before and after various mitigation strategies were analyzed that was important, it was determined that this data was adequate at the time of the 2006 study. For the purposes of this study, the shapefile previously used for the damage analysis did not cover the entire expanded study area necessitating the development of a new method to assess flood damages. Additionally, while the point shapefile was the best available data for use at the time of that analysis, there has since been the development of better GIS datasets that allow for more accurate estimation of building locations and valuations. Specifically, GIS data was acquired from the Cameron County Appraisal District for all parcels within Cameron County. This data included assessed building valuations for most properties throughout the County. The specific methodology employed to complete the damage assessment along with further discussion of the benefits that this dataset provided over that of the previous study will be discussed in Section 2.2.11.

2.2 Description of Modeling Technique and Damage Assessment

All modeling of the hydrologic and hydraulic behavior of the drainage systems discussed in this study was performed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS). HEC-HMS was used to model the hydrology of the expanded planning area, including the URRV, LRRV and CCDD3 watersheds, which is consistent with the approach taken to model CCDD1 and NMD in the 2006 study. The RDLG and TR watersheds from the 2006 study used a distributed hydrologic model called Vflo. For this study, the hydrologic models from those two systems were converted to HEC-HMS format and updated with the latest landuse data available. For all watershed models the Clark Unit Hydrograph Method was utilized which relies on the time of concentration (TC) of each subwatershed area and a storage coefficient (R) to compute a unit hydrograph. The storage routing method used for all watershed models was the modified puls routing method. This method incorporates user-specified storage-outflow relationships that were developed through an iterative process using the HEC-HMS and HEC-RAS models to calculate storage based on the cross-sectional and culvert outflow data included in the HEC-RAS model. The hydraulic analysis for all watersheds within the planning area was completed using the HEC-RAS software. A brief description of each model along with a discussion of the hydrograph and routing methods used along with general assumptions are described in subsequent sections.

2.2.1 ArcMap 9.3

ArcGIS Desktop is a comprehensive collection of professional Geographic Information Systems (GIS) applications used to solve problems, increase efficiency, make better decisions and to communicate, visualize, and analyze geographic information. In performing this work, GIS users:

- Create and use maps
- Compile, edit, and maintain geographic data
- Automate work tasks, analyze and model with geoprocessing
- Visualize and display results in maps, 3D views and dynamic, time-based displays
- Manage and maintain multiuser geographic databases
- Offer GIS resources and results to an extensive range of users for a large number of applications
- Build custom applications to share GIS
- Document and catalogue results - geographic datasets, maps, globes, geoprocessing scripts, GIS services, applications

ArcGIS Desktop is the main program for GIS professionals to manage their GIS workflows and projects and to build data, maps, models, and applications. GIS is the foundation to coordinate GIS across organizations and onto the Internet. ArcGIS Desktop includes a collection of applications including ArcCatalog, ArcMap, ArcGlobe, ArcScene, ArcToolbox, and ModelBuilder. Using these applications and interfaces, users can perform any GIS task, from simple to advanced. ArcGIS Desktop has different levels of complexity, and can address the needs of many types of users.

Additional capabilities can be added through a series of ArcGIS Desktop extension products from ESRI and other organizations. Specific extensions that were used in this study to develop hydrologic and hydraulic models are discussed in Sections 2.2.2-2.2.7.

2.2.2 HEC-GeoHMS Extension

The Geospatial Hydrologic Modeling System (HEC-GeoHMS) is an extension that is used in conjunction with Spatial Analyst within ArcMap. It aids in the development of hydrologic modeling inputs and in completing watershed delineations. 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 datasets that can then be imported into a hydrologic model such as HEC-HMS.

2.2.3 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) in ArcInfo TIN (Triangulated Irregular Network) format a HEC-RAS import file is created that contains geometric attribute data as well as many other complementary data sets. 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 including water surface profile data and velocity data can then be exported back to HEC-GeoRAS and used in conjunction with the Spatial Analyst extension for floodplain mapping.

2.2.4 Spatial Analyst Extension

ArcView's Spatial Analyst extension offers 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
- Find suitable locations and the best path between locations
- Perform statistical analysis based on the local environment, small neighborhoods or predetermined zones
- Interpolate data values for a study area based on samples
- Perform distance and cost-of-travel analyses
- 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
- Clean up a variety of data for further analysis or display

2.2.5 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.

2.2.6 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.

2.2.7 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.

2.2.8 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. This section contains a brief description of the capabilities and usage of the program; specific information is given on the selection and application of the various methods offered.

The main advantage of the HEC-HMS software is that it is free, publicly available software that has been widely and successfully used across the country since the late 90s (and its predecessor HEC-1 was developed in the 70s). It allows for the use of multiple hydrograph and routing methods to accommodate numerous watershed conditions. Another 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 easily accessed by the main screen, called the Project Definition screen.

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.

For this study the Clark hydrograph method was used. 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 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 the NLCD dataset discussed in section 2.1 and through examination of aerial photographs of the watershed. Percent conveyance is assumed to be 95% for all of Brownsville. The TC&R values computed for each subwatershed area is presented in the sections 4 and 5 along with the discussion of each watershed analysis. Base flow was assumed to be negligible and an initial and constant loss rate of 0.75 in/hr and 0.075 in/hr respectively were assumed given the clay-rich soils that are typical of the region.

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. For the purpose of this study the Modified Puls method was used.

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 Meteorological Model contains the precipitation data, either historical or hypothetical, for the HEC-HMS model. For this study frequency storms were used for a 100-yr, 50-yr, 25-yr, 10-yr, 5-yr and 2-yr storm events corresponding to expected frequencies of 1%, 2%, 4%, 10%, 20%, and 50% respectively. For further discussion of the rainfall data used see section 2.1.4.

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.

Together, the basin model, the meteorological model and the control specifications are used in conjunction with one another and the model is run to arrive at computed flows in cubic feet per second (CFS). These flows are then used as an input to the HEC-RAS model which is described in Section 2.2.9.

2.2.9 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 then be used for floodplain delineation. HEC-RAS 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.

HEC-RAS also includes the ability to model inline weirs and gates and multiple culvert openings, and is able to model 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 using the flows that are computed by the HEC-HMS model as described in Section 2.2.8 (Haestad, 2003). All networks modeled in this analysis assumed a subcritical flow regime and a normal depth boundary condition of 0.1% except in the case of the storm surge analysis where a known depth of 28-ft was assumed as discussed in Section 5.0.

2.2.10 Full Development Analyses

To evaluate the impacts of a given watershed area being fully developed without implementing runoff control policies thus allowing the overall overland flow rate from a given rainfall event to increase, flow rates were estimated based on the results of the 2006 study. The flows under the existing development scenario were observed for the CCDD1 and NMD watersheds and compared to that of the full development analysis. The impact of allowing full development without runoff controls on the NMD watershed resulted in an increase of flows of less than 5%. The relatively low impact on flow rates is due to the fact that at the time of the study, the NMD watershed was already close to being fully developed. The analysis of the CCDD1 watershed revealed in increase of flow rates of approximately 20-25%. The CCDD1 watershed was still relatively undeveloped at the time of the study resulting in the larger increase of flow rates due to the impact of urbanization. The three additional watersheds that were evaluated in this study: CCDD3, URRV, and LRRV are still predominately undeveloped. As such, it was determined that the % flow increases that resulted in the CCDD1 watershed would be more representative of the likely impact that urbanization would have on the expanded study area. Based on this assumption, to model the impacts of full development on the new watershed areas in this study, it was assumed that all existing flows would be increased by 25%. The results of this analysis will be discussed in subsequent sections

2.2.11 Damage Assessment Technique

To estimate flood damages from a given frequency storm event the Cameron County Appraisal District parcel data described in Section 2.1.5 was utilized in conjunction with the Brownsville Public Utilities Board (BPUB) point shapefile identifying buildings in Brownsville (also discussed in Section 2.1.5). As mentioned in Section 2.1.5 the parcel data was preferred over the building point file due to completeness with respect to the study area (the point file did not cover the entire study area) and improved accuracy in building value assessments. However, the issue with relying on this dataset alone is that the parcels define the area of an entire property, whereas this analysis is only concerned with the portion of the parcel that a building is located. Also, in some situations there are parcels with multiple buildings. In this regard it is preferential to use polygons representing the footprint of the building only. However, since this data is not available and would be extremely time and cost prohibitive to develop for the purpose of this analysis, points representing the location of the building within the parcel was the next best option.

To resolve the issues presented regarding both the parcel data and the building point file, a hybrid point file was created. The BPUB building file was used as the primary file and the parcel file was used as a secondary file. The steps below describe the process used to develop this file.

1) Using ArcMap 9.3 software the parcel data was used to create a point file to define those parcels that contained buildings on site. The Xtools Shapes to Centroids tool in ArcMap 9.3 was used to

place a point at the centroid of every parcel that had a building onsite. To determine which parcels contained buildings, the “year built” and “living area” attributes within the parcel database were examined. To create the building point file, only those parcels that contained a value for “year built” and had a “living area” value greater than zero were included. The main issue with this point file is the assumption that the building was located at the centroid of the parcel. While this assumption sometimes holds true it is often not the case. In contrast, the point file developed by the BPUB places the points on top of the actual building. For this reason, the point file created from the parcel file was considered a secondary file to the BPUB point file for the purpose of geographically locating the buildings within the watershed.

2) A spatial join was performed between the point file created in step 1 and the BPUB point file. The purpose of the join was to incorporate the building valuation data from the parcel data to the BPUB shapefile, for those portions of the study area that had data coverage from this file, and to add the parcel centroid points for those portions of the study area that were not covered by the BPUB point file.

3) Upon completion of the spatial join in step 2, all buildings that did not have valuations assigned to them in the parcel database were given an assumed value of $60,000 as was done in the 2006 analysis.

4) To remedy the issue discussed in section 2.1.5 regarding points being placed on structures with a relatively small value compared to that of a primary-type building (i.e. homes, businesses) such as a gazebo, garage or tool shed, the “building area” attribute in the joined point/parcel database file was examined to identify all structures with a building area of 400-square feet or less. Upon close examination of several small gazebo and garage-type structures, it was determined that 400-square feet was a reasonable value to separate homes and businesses from gazebos, garages, and tool sheds.

5) The last step was to examine those parcels that had multiple buildings associated with them. In these cases, the appraisal district’s parcel data contained only one valuation to represent the entire parcel and not each individual building. Furthermore, because it is possible for one building on a parcel to be impacted by flooding and another building not, it was necessary to assign separate values to each building on a parcel. In these situations it was assumed that each building would be valued at $60,000 as done in the 2006 study.

The point file that resulted from steps 1-5 above was used to estimate damages from a 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year frequency rainfall event. The floodplains and depth grids that were delineated using the HEC-GeoRAS extension in ArcMap as discussed in Section 2.2.3 were overlain on the point value and those points that intersected with the floodplain were assigned an assumed damage based on the predicted depth of flooding at that site and the overall valuation of the building. The relationship between flood depths and the percent damage to a given building is displayed in Table 2-3 below. Overall damages from any given storm event were calculated using the following formula:

Damages = % Damage of Building Based on Inundation Depth (from Table 2-3) x Building Value

Inundation Depth (ft)	Damage Percentage
0-0.5	15%
0.5-1	18%
1-1.5	20%
1.5-2	24%
2-2.5	27%
2.5-3	27%
3-3.5	31%
3.5-4	31%
4-4.5	31%
4.5-5	31%
5.0-8	44%
8.0-12	48%
>12.0	50%

Table 2-3. Relationship between inundation depth and assumed percent damage to building

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 (discussed in Section 6.3), 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.

3.0 Flooding Analysis of Resaca del Rancho Viejo (RRV) and Cameron County Drainage District No. 1 Ditch No. 3 (CCDD3)

The HEC-HMS and HEC-RAS software described in Sections 2.2.8 and 2.2.9 were used to develop hydrologic and hydraulic models to evaluate existing, or baseline conditions of both the Resaca del Ranch Viejo (RRV) and Cameron County Drainage District No. 1 Ditch No. 3 (CCDD3) systems. Originally, it was thought that the RRV functioned as one system and was intended to be set up as one single model. However, field reconnaissance and data collection revealed that the Resaca was actually split into two separate systems, upper and lower. The upper portion of RRV is split from the lower section by an overflow structure that routes flow from the upper RRV (URRV) to CCDD3. As such the URRV was modeled with one HEC-HMS model and the lower RRV (LRRV) was modeled as a separate system. To best capture the inter-related nature between URRV and CCDD3, the two systems were merged during the hydraulic analysis in a single HEC-RAS model. The results of each modeling effort are described throughout the rest of Section 3.

3.1 Upper RRV and CCDD3

3.1.1 Description of URRV and CCDD3 Watershed Models

The upper segment of RRV (URRV) begins near FM 732 and follows a sinuous path that generally flows in a northwest to southeast direction. The Resaca is approximately 25 river miles in length and outfalls into CCDD3 through an overflow structure separating the upper segment of RRV from the lower segment discussed in Section 1.2. The CCDD3 ditch is approximately 7 miles long and outfalls into Cameron County Drainage District No. 1 Ditch No. 2. These two watersheds were delineated separately but merged to complete the hydraulic HEC-RAS analysis since the flow from URRV drains directly into CCDD3. The URRV watershed is approximately 29,406 acres (45.9 square miles) and the CCDD3 watershed is approximately 8,664 acres (13.5 square miles) for a combined watershed area of 38,070 acres (59.5 square miles).

The URRV Watershed was subdivided into 48 subwatersheds and CCDD3 into 19 subwatersheds as displayed in Figure 3-1. Within the entire combined watershed area typical slopes are less than 1%. The majority of the area is undeveloped with only approximately 20% of the total area currently developed. The majority of the land use throughout the combined watershed area is characterized as agricultural fields with regions of residential development directly adjacent to the Resaca, predominately in the Town of Rancho Viejo. The computed Clark Method TC&R values used in the hydrological model for each watershed are displayed in Tables 3-1 & 3-2. To simulate the individual resaca "pools" and the function of the culverts that connect individual Resaca pools 62 reach segments were used in the HEC-HMS model for the URRV watershed each one including storage-routing information to represent the stormwater flow through the system. For the CCDD3 HEC-HMS model there were 19 routing reaches developed to model the flow of stormwater through the ditch.

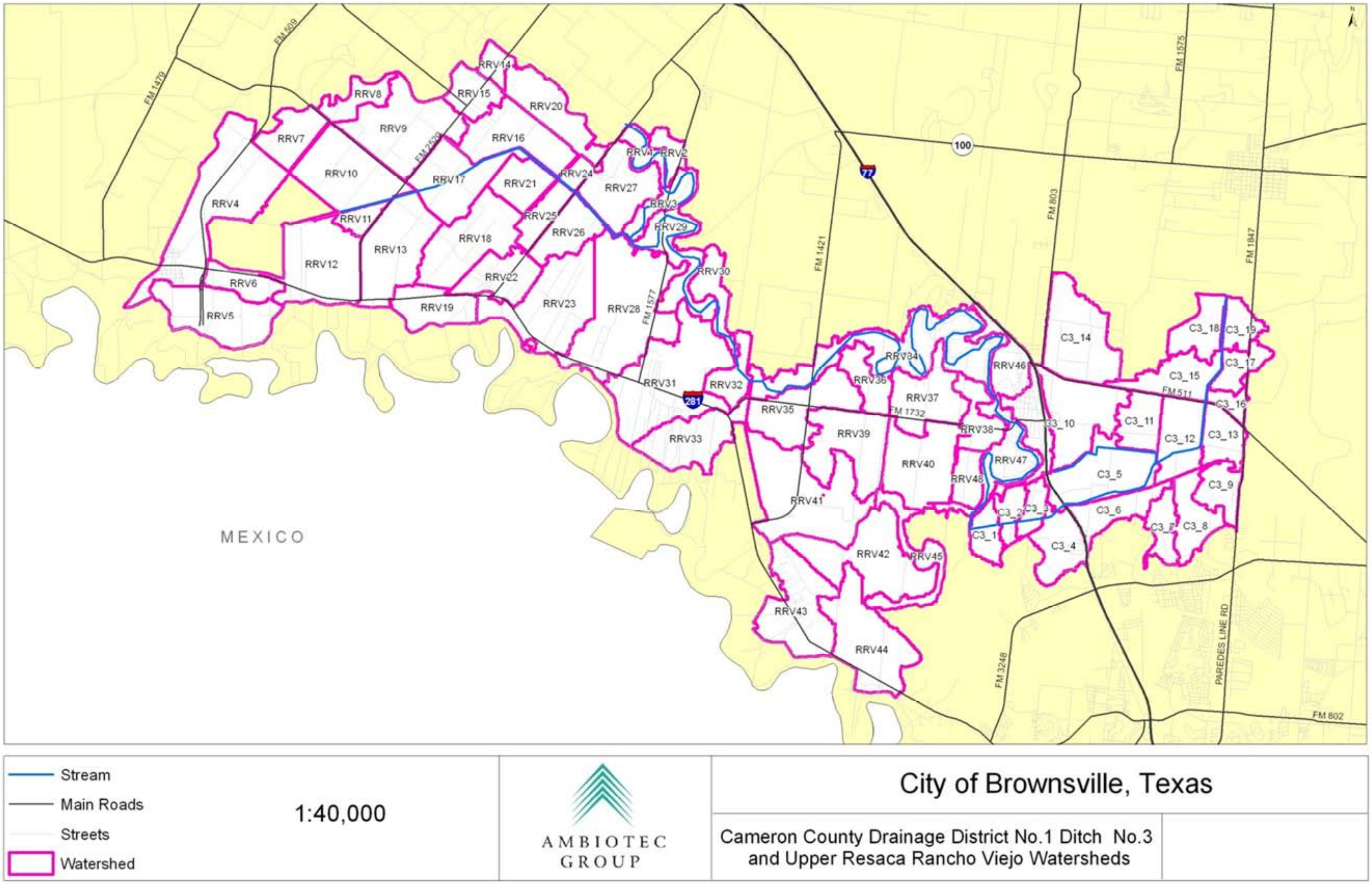

Figure 3-1. Subwatershed map for CCDD3 and URRV

Subareas	Drainage Area (Acres)	TC (Hours)	R (Hours)
RRV1	183.2	32.70	31.79
RRV2	162.7	1.05	8.34
RRV3	285.7	1.78	14.30
RRV4	1469.7	3.79	14.93
RRV5	802.1	0.92	7.47
RRV6	284.0	0.51	7.20
RRV7	386.3	2.47	7.57
RRV8	273.7	3.03	10.65
RRV9	989.3	4.29	9.99
RRV10	862.4	3.48	11.02
RRV11	162.5	0.39	8.07
RRV12	949.4	2.34	9.72
RRV13	894.1	2.37	12.84
RRV14	152.9	0.73	6.26
RRV15	313.1	1.36	8.14
RRV16	699.5	2.36	10.75
RRV17	714.6	3.68	12.71
RRV18	867.5	4.54	14.66
RRV19	436.9	1.04	7.42
RRV20	586.6	2.64	12.41
RRV21	345.1	1.39	7.92
RRV22	466.1	1.91	9.49
RRV23	1055.7	4.27	11.45
RRV24	109.6	0.68	5.19
RRV25	257.5	1.40	9.57
RRV26	402.7	1.52	9.67
RRV27	638.8	1.53	9.22
RRV28	1531.2	2.75	12.19
RRV29	578.5	5.23	16.61
RRV30	308.0	2.10	9.42
RRV31	1398.4	1.46	10.04
RRV32	268.8	0.70	7.30
RRV33	650.0	2.57	11.05
RRV34	1375.8	21.55	20.23
RRV35	724.4	2.35	10.45
RRV36	308.5	2.05	8.37
RRV37	618.6	1.82	8.39
RRV38	337.0	2.04	8.37
RRV39	788.9	7.00	17.04
RRV40	816.4	4.14	12.02
RRV41	1295.9	2.95	15.31
RRV42	1011.4	1.99	11.74
RRV43	879.5	4.15	10.41
RRV44	945.4	0.68	12.30
RRV45	380.3	1.63	13.68
RRV46	360.6	2.98	10.00
RRV47	540.4	3.47	14.98
RRV48	235.9	0.45	7.06

Table 3-1. Upper Rancho Viejo TC&R Values for existing conditions

Subareas	Drainage Area (Acres)	TC (Hours)	R (Hours)
C3_1	233.0	0.2672676	5.786161
C3_2	195.8	0.1594217	5.159644
C3_3	179.8	0.2528004	5.013442
C3_4	438.4	1.0490336	7.040985
C3_5	779.5	1.4508715	8.862335
C3_6	369.9	1.2553488	6.83741
C3_7	262.4	0.9177426	5.515519
C3_8	414.1	2.0633834	8.5059
C3_9	297.0	0.8124119	4.136505
C3_10	1190.4	0.8063057	6.763085
C3_11	348.2	1.4740861	5.915032
C3_12	513.9	1.5582053	7.107452
C3_13	332.2	1.1080788	6.281025
C3_14	878.7	1.9595784	7.191886
C3_15	471.0	3.5853633	9.436683
C3_16	131.8	0.4290021	5.341233
C3_17	251.5	1.347235	6.326255
C3_18	366.1	1.5271682	7.267659

Table 3-2. Cameron County District no.1 Ditch 3 TC&R Values for existing conditions

The HEC-RAS models for each of the two watersheds (URRV and CCDD3) were merged due to the inter-connected nature of the two watersheds. To develop the HEC-RAS model the HEC-GeoRAS extension in ArcMap 9.3 was used to develop a RAS import file. The RAS import file provides the model with geometric data to describe the storage in a channel. Combined the two systems were characterized by over 180 cross sections providing the model with elevation data, roughness coefficients, bank locations and distances. Culvert data for 25 structures was added to the model manually based on the culvert data that was collected as described in Section 2.1.3. Culverts that were incorporated into the model for each watershed are displayed in Tables 3-3 & 3-4.

Street Name	Station	Type	Number	Size
FM 1577 (1st crossing)	225700	RCP	1	3-ft
FM 1577 (2nd crossing)	218800	RCP	1	3-ft
Island States St.	213300	RCP	1	2-ft
FM 1577 (3rd crossing)	209000	RCP	1	2-ft
Barreda Gardens Rd.	179500	RCP	2	4-ft
Grove Park Rd.	171600	CMP	1	5-ft
Avenida Escandon	161000	Bridge	1	N/A
Taco St.	158500	Bridge	1	N/A
Enchilada St.	156500	Bridge	1	N/A
Carment Ave.	152600	Bridge	1	N/A
Rancho Viejo Drive	149670	Bridge	1	N/A
Inca St.	147000	Bridge	1	N/A
Balboa Avenue	145000	Bridge	1	N/A
Bolivar Avenue	138700	Bridge	1	N/A
El Dorado Avenue	136000	Dam	1	N/A
FM 1732 / Cavazos-Olmito Rd.	128900	RCP	1	5-ft
Overflow Structure to CCDD3	110740	weir/RCP	1	3-ft

Table 3-3. Culverts in the Upper Resaca del Rancho Viejo Watershed

Street Name	Station	Type	Number	Size
West Drainage Ditch	31000	Box	1	4-ft x 7-ft
Butler St.	30000	CMP	1	7.5-ft
Upstream of RR	27400	Box	2	4-ft x 7-ft
RR near Flea Market 1	26825	Box	1	6-ft x 6-ft
RR near Flea Market 2	26740	Box	1	15-ft x 7-ft
US 77/83	25300	Box	1	7-ft x 10-ft
Private Crossing	15800	Box	1	7-ft x 10.5-ft
FM 511	9000	Bridge	1	N/A

Table 3-4. Culverts in the Cameron County Drainage District No. 1 Ditch No. 3 Watershed

3.1.2 Upper RRV and CCDD3 Existing Development Analysis

The results of the existing development analysis on the URRV/CCDD3 watershed reveal that a 10-yr frequency rainfall event would result in approximately 17% of the entire watershed area (6,426 acres) being inundated with water. This level of flooding is predicted to result in the flooding of approximately 360 buildings. For the 100-yr frequency event this value jumps to nearly 28% (10,326 acres) and the flooding of over 560 buildings. The overall number of buildings flooded along with the expected depth of flooding for both the 10-yr and 100-yr rainfall events is presented in Table 3-5. As mentioned in Section 2.1.4, it is important to note that these values assume that the rainfall total for a given frequency event is experienced area-wide across the entire watershed. The floodplain delineations for each of these events may be viewed in Figures 3-2 & 3-3. Floodplain maps for the other rainfall frequency events (2-yr, 5-yr, 25-yr and 50-yr) may be viewed in Appendix A. The computed water surface elevations (W.S.E's) for both the 10-yr and 100-yr rainfall events are presented in Table 3-6 and Figure 3-4. For each of these events large areas of inundation may be observed in the northwest portion of URRV, the area just south of the Town of Rancho Viejo, and a large area surrounding the downstream end of CCDD3. While the downstream end of CCDD3 and the area south of the Town of Rancho Viejo is largely undeveloped, there are some residential type areas in the Northwest Portion of the URRV watershed north of Military Highway between County Rd. 732 and County Rd. 1577, outside of the City of Brownsville limits.

	Existing Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	100	139
0.5-1	62	108
1-1.5	47	65
1.5-2	34	52
2-2.5	33	47
2.5-3	34	36
3-3.5	26	39
3.5-4	13	33
4-4.5	7	23
4.5-5	4	12
5.0-8	0	8
8.0-12	0	0
>12.0	0	0
Total	360	562

Table 3-5. Number of Structures/Buildings predicted to be flooded in the URRV/CCDD3 watershed area under existing conditions for the 10-yr and 100-yr frequency rainfall

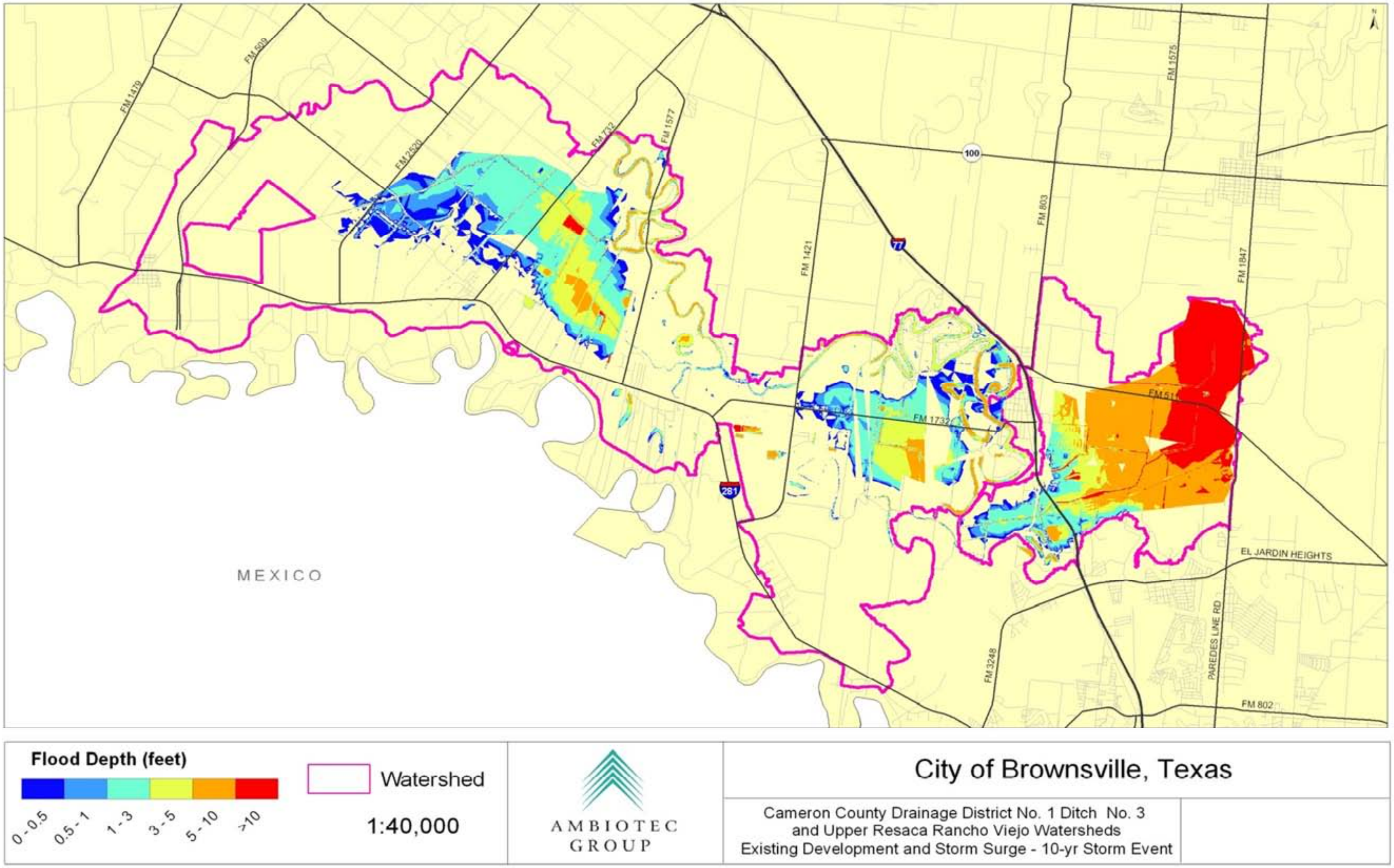

Figure 3-2. Existing development floodplain for the URRV/CCDD3 watersheds assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

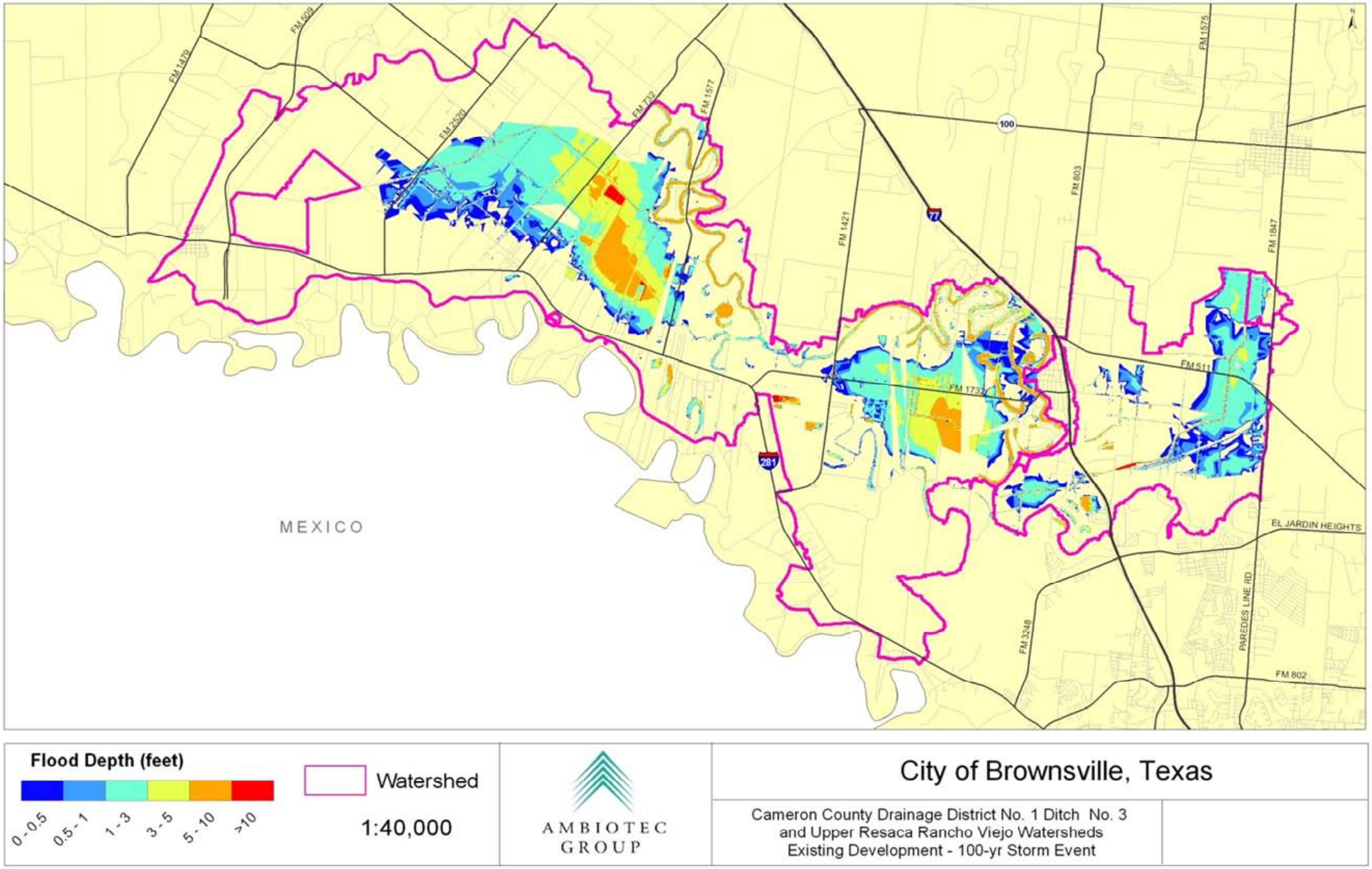

Figure 3-3. Existing development floodplain for the URRV/CCDD3 watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
FM 732 (URRV)	236667.20	10-yr	19.3	43.42
	236667.20	100-yr	33.3	44.38
FM 1577 (1st crossing)(URRV)	225636.00	10-yr	69.4	43.41
	225636.00	100-yr	109.6	44.37
Rice Tract Rd. (URRV)	202893.80	10-yr	1321.7	39.29
	202893.80	100-yr	2894.6	41.75
Near Guagolota Rd. (URRV)	188108.40	10-yr	1062	36.16
	188108.40	100-yr	2384.7	36.9
FM 1421 / Barreda Garden Rd. (URRV)	179139.60	10-yr	1075.4	35.11
	179139.60	100-yr	2452.2	36.57
Escandon Ave. (URRV)	160701.80	10-yr	798.1	34.41
	160701.80	100-yr	1804.2	35.19
Balboa Rd. (URRV)	149664.30	10-yr	816	34.3
	149664.30	100-yr	1881.4	34.72
FM 1732 / Cavazons Olmito Rd. (URRV)	128689.10	10-yr	772.6	31.21
	128689.10	100-yr	1875	32.61
Near Lakeside Blvd. 1 (URRV)	120539.90	10-yr	772.6	31.17
	120539.90	100-yr	1875	32.47
Near Lakeside Blvd. 2 (from Resaca to Ditch)	111686.00	10-yr	2308.4	30.51
	111686.00	100-yr	3917.9	31.16
Before Overflow Structure to CCDD3	110760.00	10-yr	2308.4	29.54
	110760.00	100-yr	3917.9	30.12
Railroad (CCDD3)	27014.17	10-yr	2491.8	25.06
	27014.17	100-yr	4306.8	25.64
US 77/83 (CCDD3)	25093.72	10-yr	2573.4	21.29
	25093.72	100-yr	4476.9	23
Old Alice Rd (CCDD3)	22959.53	10-yr	2573.4	20.46
	22959.53	100-yr	4476.9	21.27
Box Culvert after Ditch Junction (CCDD3)	15679.25	10-yr	1057.4	18.41
	15679.25	100-yr	1638.7	18.79
FM 511 (CCDD3)	8888.71	10-yr	1746	17.07
	8888.71	100-yr	2798.5	17.56
Undeveloped Area Before Outfall	5438.58	10-yr	2169	15.94
	5438.58	100-yr	3579.5	16.46
Outfall	317.46	10-yr	2167.9	12.36
	317.46	100-yr	3638.8	12.84

Table 3-6. URRV/CCDD3 existing development conditions flows (Q) and predicted water surface elevations (W.S.E)

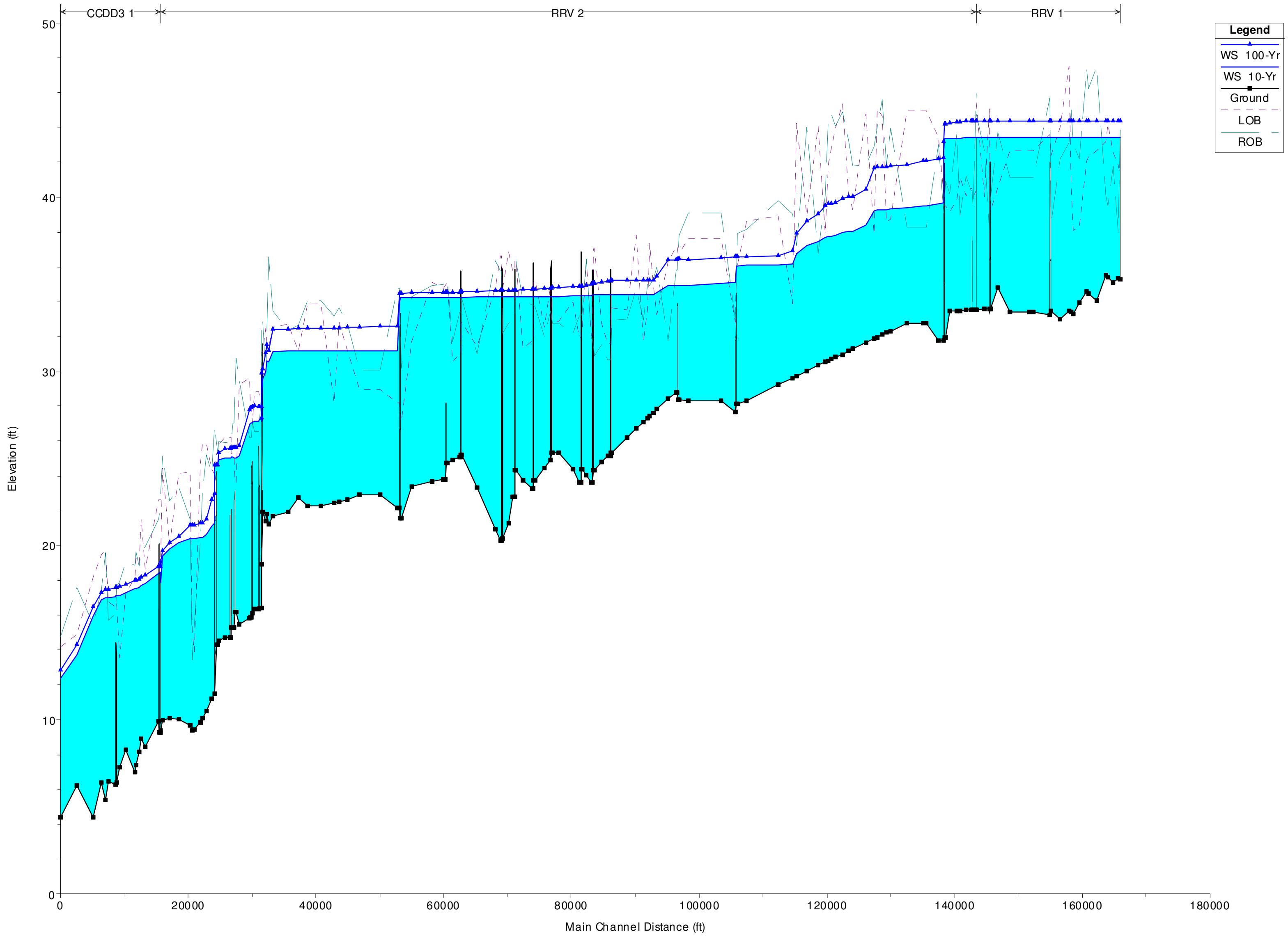

Figure 3-4. Water surface elevation profile for URRV/CCDD3 under existing conditions

3.1.3 Upper RRV and CCDD3 Full Development Analysis

As discussed during the 2006 study, the impacts of urbanization on drainage patterns can be significant. The increase in impervious surfaces that accompanies urbanization and development generally increases both runoff volumes and peak flow rates (Bedient and Huber, 2002). As a result of this occurrence, one of the major recommendations from the 2006 study was to implement runoff controls that would require that existing levels of runoff from a given site be maintained at existing development conditions as development ensues. It is much more cost-effective than implementing major mitigation strategies to fix a flooding issue in the future and as such is an essential strategy to prevent future flooding.

The results of the full development analysis for the URRV/CCDD3 watershed revealed that with an assumed 25% increase in flow, the 10-yr rainfall event would result in roughly 24% of the entire watershed area (8896 acres) being inundated with water and the 100-yr event would cause over 30% inundation with floodwaters (11,264 acres). The number of buildings predicted to be flooded under this scenario is estimated at 403 for the 10-yr rainfall event and 660 for the 100-yr, again assuming that the given rainfall totals are experienced across the entire watershed area. The number of buildings predicted to flood within a given water depth for both the 10-yr and 100-yr frequency storm is presented in Table 3-7. The resulting floodplains from each of these events may be observed in Figures 3-5 & 3-6. Predicted flows and water surface elevations (W.S.E.'s) from this analysis may be viewed in Table 3-8 and Figure 3-7. Overall, the computed water surface elevations increased between 0.1 to 1 foot for either the 10-yr or the 100-yr for the full development scenario as compared to the existing development conditions. The floodplains for the other rainfall frequency events (2-yr, 5-yr, 25-yr, and 50-yr) may be viewed in the appendix.

	Full Dev. Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	116	150
0.5-1	66	116
1-1.5	50	97
1.5-2	39	49
2-2.5	33	59
2.5-3	36	42
3-3.5	30	42
3.5-4	18	35
4-4.5	9	33
4.5-5	4	22
5.0-8	2	15
8.0-12	0	0
>12.0	0	0
Total	403	660

Table 3-7. Number of Structures/Buildings predicted to be flooded in the URRV/CCDD3 watershed area under assumed full development conditions for the 10-yr and 100-yr frequency rainfall

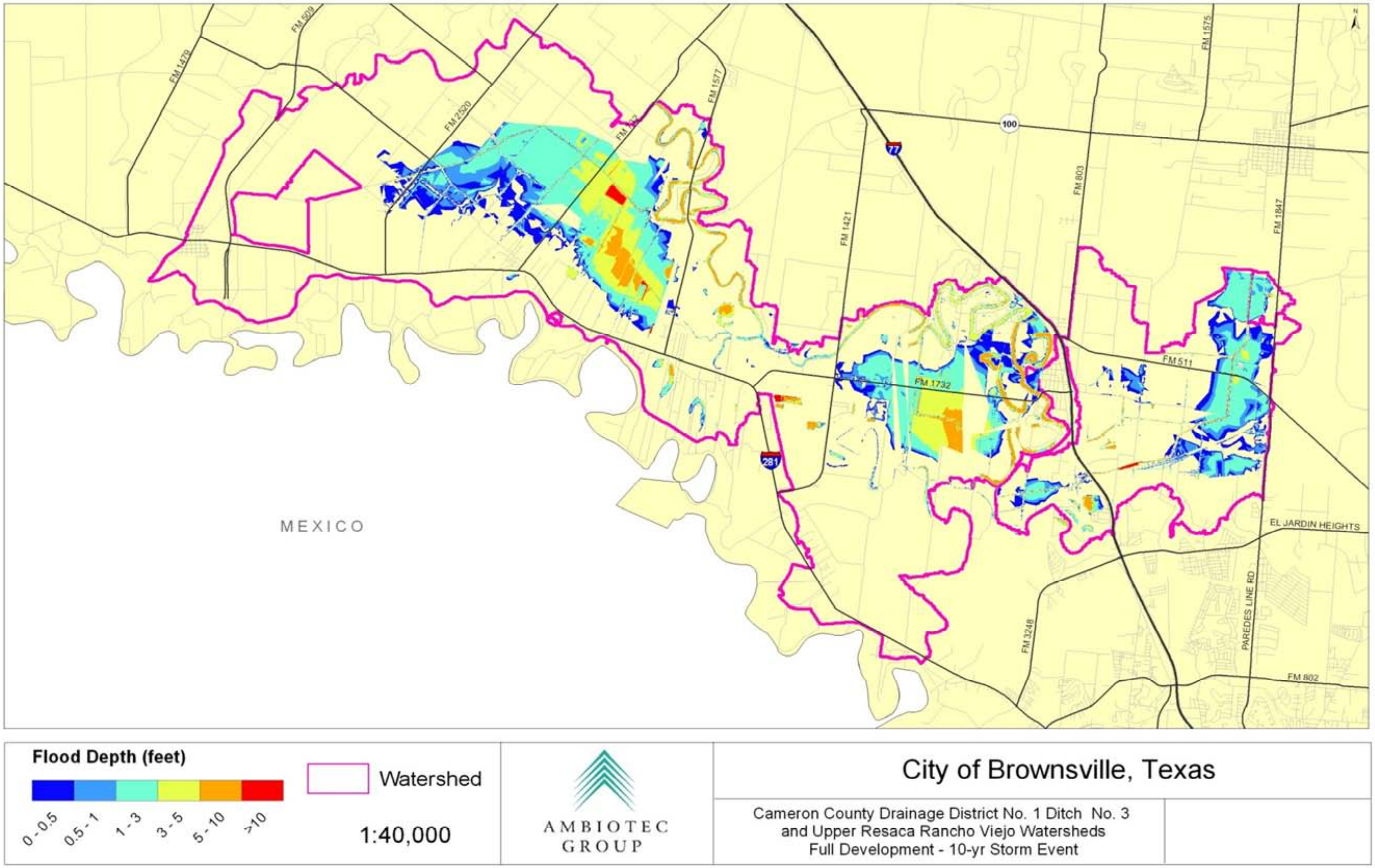

Figure 3-5. Full development floodplain for the URRV/CCDD3 watersheds assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

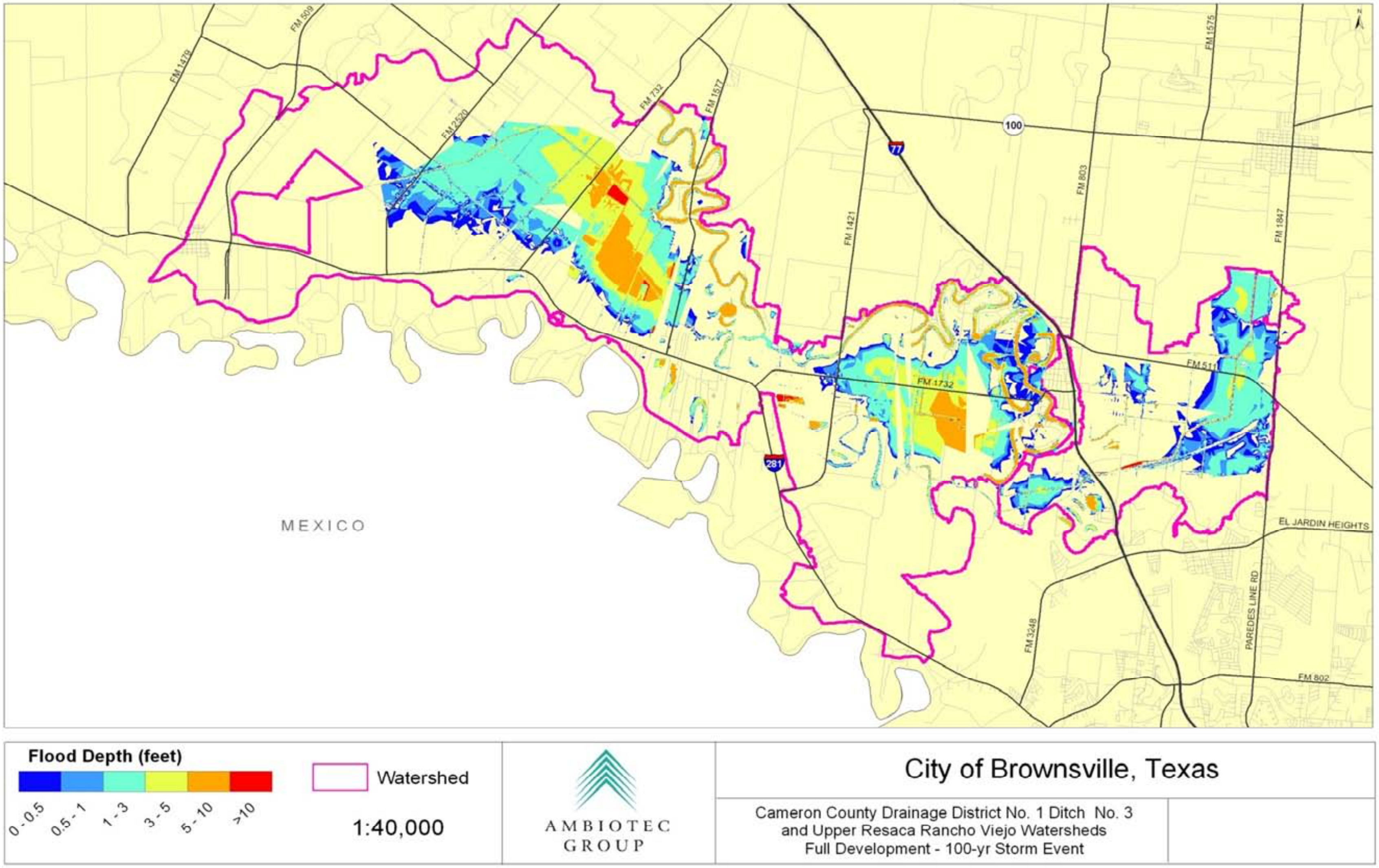

Figure 3-6. Full development floodplain for the URRV/CCDD3 watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
FM 732 (URRV)	236667.20	10-yr	24.1	43.67
	236667.20	100-yr	41.6	44.74
FM 1577 (1st crossing)(URRV)	225636.00	10-yr	86.8	43.67
	225636.00	100-yr	137	44.74
Rice Tract Rd. (URRV)	202893.80	10-yr	1652.1	39.94
	202893.80	100-yr	3618.3	42.54
Near Guagolota Rd. (URRV)	188108.40	10-yr	1327.5	36.27
	188108.40	100-yr	2980.9	37.56
FM 1421 / Barreda Garden Rd. (URRV)	179139.60	10-yr	1344.3	35.45
	179139.60	100-yr	3065.3	37.16
Escandon Ave. (URRV)	160701.80	10-yr	997.6	34.56
	160701.80	100-yr	2255.3	35.55
Balboa Rd. (URRV)	149664.30	10-yr	1020	34.38
	149664.30	100-yr	2351.8	34.89
FM 1732 / Cavazons Olmito Rd. (URRV)	128689.10	10-yr	965.8	31.74
	128689.10	100-yr	2343.8	33.26
Near Lakeside Blvd. 1 (URRV)	120539.90	10-yr	965.8	31.68
	120539.90	100-yr	2343.8	33.08
Near Lakeside Blvd. 2 (from Resaca to Ditch)	111686.00	10-yr	2885.5	30.83
	111686.00	100-yr	4897.4	31.5
Before Overflow Structure to CCDD3	110760.00	10-yr	2885.5	29.73
	110760.00	100-yr	4897.4	30.37
Railroad (CCDD3)	27014.17	10-yr	3114.8	25.29
	27014.17	100-yr	5383.5	25.97
US 77/83 (CCDD3)	25093.72	10-yr	3216.8	21.87
	25093.72	100-yr	5596.1	23.99
Old Alice Rd (CCDD3)	22959.53	10-yr	3216.8	20.75
	22959.53	100-yr	5596.1	21.71
Box Culvert after Ditch Junction (CCDD3)	15679.25	10-yr	1321.8	18.6
	15679.25	100-yr	2048.4	18.98
FM 511 (CCDD3)	8888.71	10-yr	2182.5	17.29
	8888.71	100-yr	3495.1	17.83
Undeveloped Area Before Outfall	5438.58	10-yr	2711.4	16.19
	5438.58	100-yr	4474.4	16.67
Outfall	317.46	10-yr	2709.9	12.55
	317.46	100-yr	4548.5	13.09

Table 3-8. URRV/CCDD3 full development conditions flows (Q) and predicted water surface elevations (W.S.E)

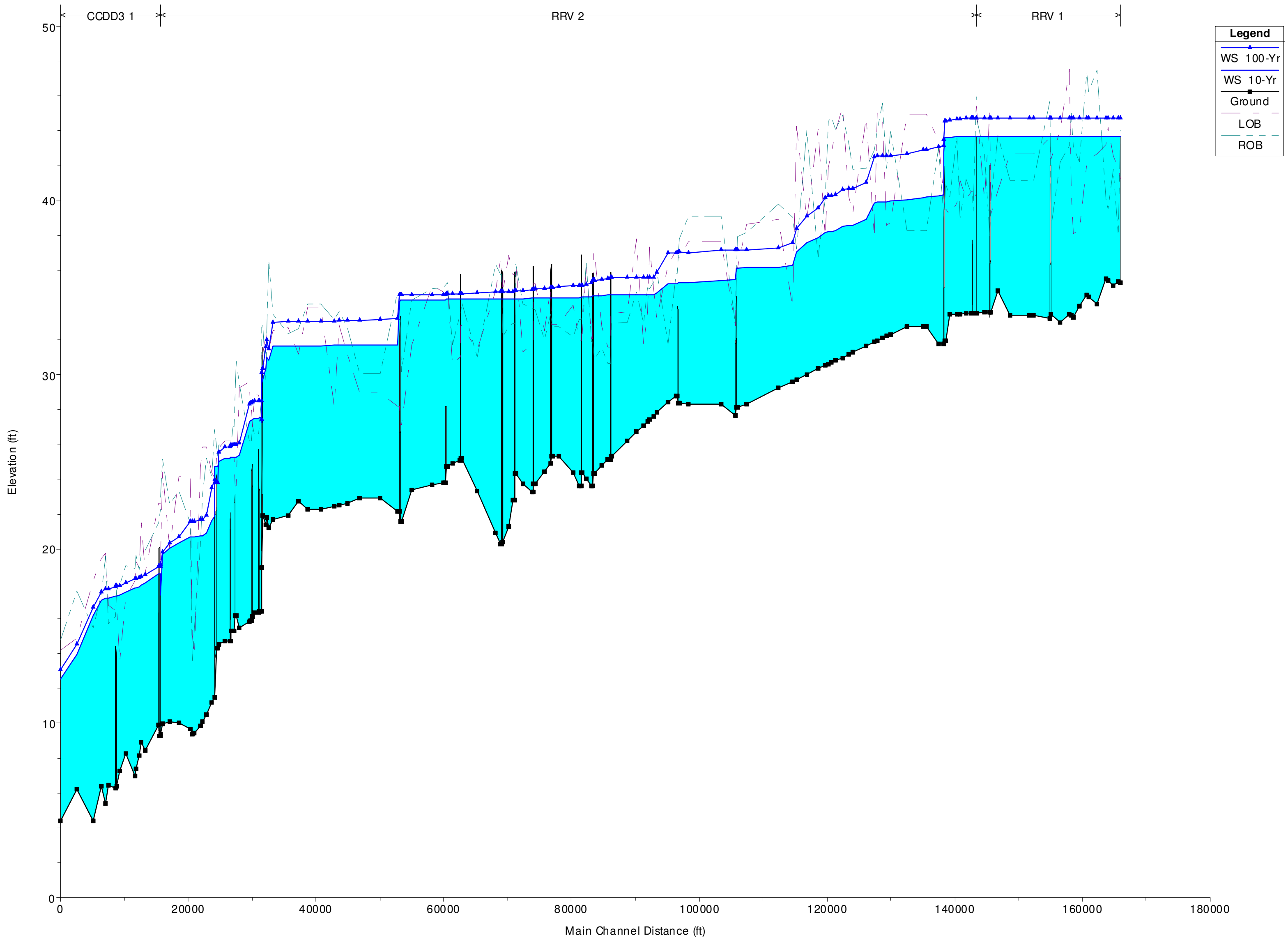

Figure 3-7. Water surface elevation profile for URRV/CCDD3 under full development conditions

3.2 Lower RRV

3.2.1 Description of LRRV Watershed

The lower segment of RRV (LRRV) begins downstream of where the overflow box that carries URRV to CCDD3 as described in Section 1.2 approximately two-miles northwest from the intersection of Alton Gloor (FM 3248)and US 77/83 (Figure1-2). The Resaca traverses in a roughly west to east direction for approximately 18.4 miles before its outfall into a drainage ditch northwest of the Port of Brownsville, eventually terminating at San Martin Lake. While the land areas immediately surrounding the Resaca banks are largely developed with residential type landuse, approximately 40% of the land area remains undeveloped. Much of the recent development in Brownsville has occurred in the Northwest quadrant of the City as much of the area south is already developed and is bound by Mexico to the South and the Gulf of Mexico to the East.

The watershed area was divided into 11 subareas as displayed in Figure 3-8 and is approximately 4,032 acres (6.3 square miles) with typical slopes less than 1.3%. The subwatersheds are labeled RRV56 – RRV61d as the upper and lower RRV watersheds were originally viewed as one single watershed and manually split after watershed delineation because of the manmade overflow structure diverting flow from the upper reach of RRV into CCDD3. Furthermore, subwatersheds RRV59 and RRV61 were manually split after the HEC-GeoRAS delineation to better represent overflow characteristics into the Resaca. The computed Clark Method TC&R values used in the hydrological model for each watershed are displayed in Table 3-9.

The HEC-RAS model for each of the LRRV watershed was again developed through the use of the HEC-GeoRAS extension in ArcMap 9.3 to create a RAS import file. The HEC-RAS model for LRRV was characterized by over 121 cross sections and 22 culvert structures. The culverts that were incorporated into the model for the LRRV watershed are displayed in Table 3-10.

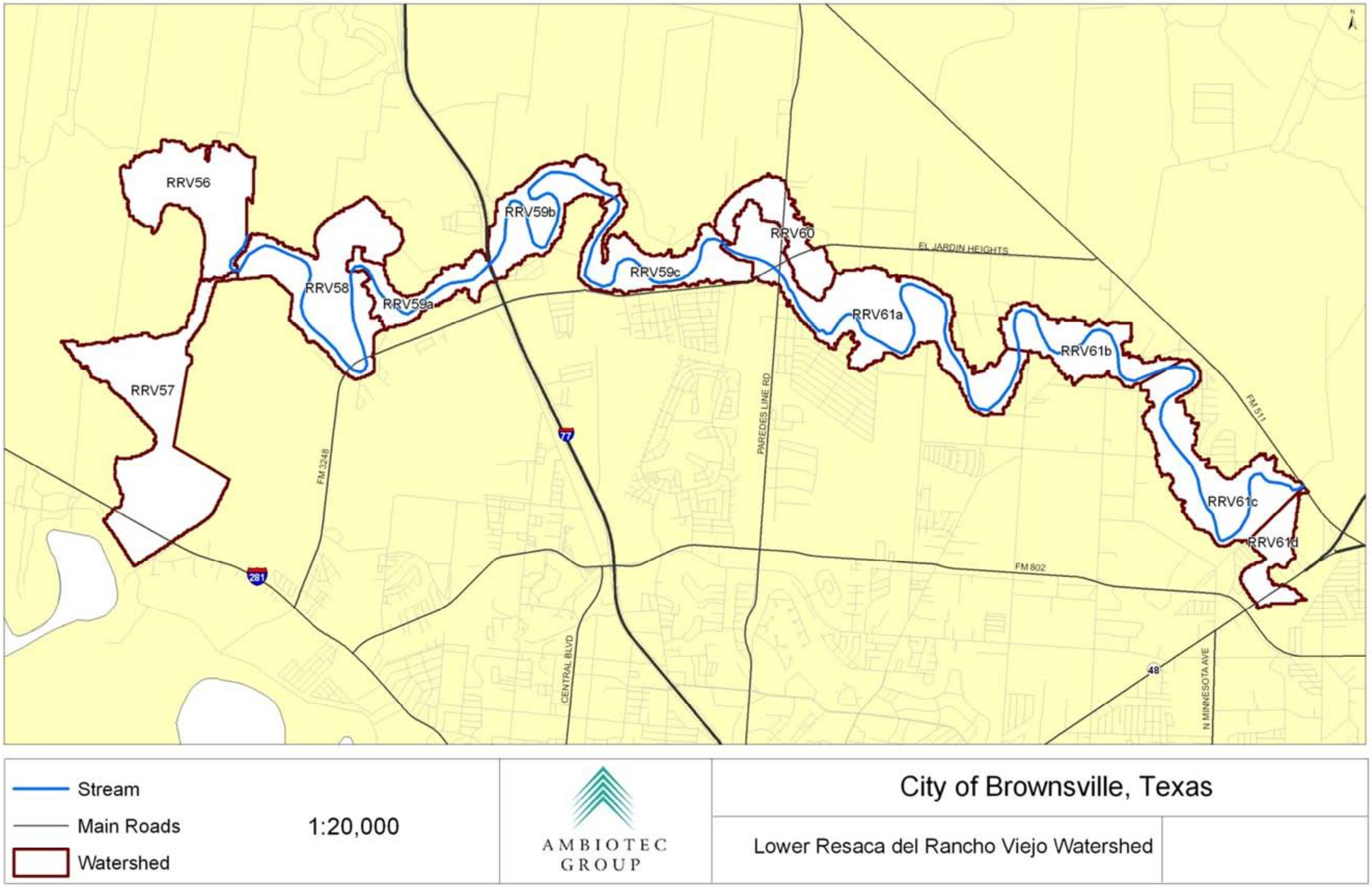

Figure 3-8. Subwatershed map for LRRV

Street Name	Station	Type	Number	Size
East of Reservoir	90600	RCP	1	2-ft
East of Reservoir 2	88900	CMP	1	4-ft
Alton Gloor 1 (1st crossing)	86850	RCP	1	5-ft
Alton Gloor 1 (2nd crossing)	85800	RCP	1	5-ft
Sandyhill and Lakeway	79500	Box Weir / RCP	1	6-ft x 8-ft, 1.5-ft
Railroad	76900	CMP	2	8.5-ft
US 77/83	74800	RCP	2	6-ft
Resaca Point Dr.	72000	Box	1	4-ft x 4-ft
Duncan Rd.	61900	RCP	1	3-ft
Rustic Manor Dr.	57500	Box	2	8-ft x 6-ft
Stagecoach Trail	53600	Box	1	8-ft x 6-ft
Btwn Stagecoach and Hike & Bike	51200	Weir	1	N/A
Upstream of Hike & Bike Overflow	49000	Box	3	5-ft x 2-ft
Paredes Line Rd. Overflow	48000	Box/RCP	1	5-ft x 2-ft, 3-ft
Dana Rd.	36200	RCP	1	5-ft
Sol Rd.	27650	RCP	1	5-ft
Robindale Rd.	25400	RCP	1	5-ft
Old Port Isabel Rd.	21500	Bridge (2 piers)	1	N/A
Charmaine Rd.	7100	RCP	1	4-ft
Heron Drive Overflow	2900	RCP	1	4-ft
FM 511 near RR (Ditch)	2350	Bridge	1	N/A
RR near Port	1200	RCP	3	(2) 5-ft, 6.7-ft

Table 3-9. Culverts in the Lower Resaca del Rancho Viejo Watershed

Historically there have been few flooding issues within this watershed. RRV, like the other area Resacas are characterized by narrow watersheds with relatively wide drainage features and high banks. In fact, the banks along area resacas are typically the highest areas throughout the City. These features provide a significant amount of stormwater storage capacity and as a result, these watersheds typically do not experience as significant of a level of flooding as other areas throughout the City. One area of concern within this watershed area has been in the vicinity of Cameron Park. Past rainfall events in the City have caused some flooding concerns upstream of a land embankment across the Resaca near Ofelia St. southeast from the intersection of Alton Gloor and Paredes Line Road. This area has experienced

Subareas	Drainage Area (Acres)	TC (Hours)	R (Hours)
RRV56	412	1.63	10.29
RRV57	660	3	12.5
RRV58	518	3.51	10.35
RRV59a	172	2.01	7.32
RRV59b	281	2.16	10.22
RRV59c	288	1.99	9.59
RRV60	190	2.17	5.76
RRV61a	651	6.97	12.99
RRV61b	267	2.08	8.03
RRV61c	436	3.49	10.56
RRV61d	132	1.08	6.53

Table 3-10. Lower Rancho Viejo TC&R Values for existing conditions

repeated out of banks flooding events encroaching in several residents backyards and in some cases reaching levels that approach finish floor elevations of several homes that were constructed within the low bank of the Resaca. To help mitigate this, a portion of the land embankment was dug out to allow flow to more easily flow downstream. It is with this remedy that the existing conditions analysis was completed.

3.2.2 Lower RRV Existing Development Analysis

The results of the existing conditions analysis reveal minimal amounts of overland flooding for any rainfall frequency event especially under the 10-yr frequency. The 10-yr and 100-yr floodplains may be viewed in Figures 3-9 &3-10, all other floodplains may be viewed in Appendix A. Upward of a 10-yr frequency rainfall event some degree of out of bank flooding is experienced in the uppermost portion of the watershed. However, this area is largely undeveloped and as such does not significantly contribute to flooding damages. The largest concern in this area is the region off of Sunset drive just north of Alton Gloor (FM 3248) before it sharply turns to the south.

In the vicinity of Cameron Park, some level of out of bank flooding is observed at all frequency events but doesn't appear to impact many homes until the between the 10-yr and 25-yr frequencies. Observation of these areas both in aerials and in the field reveal that many of the structures that have experienced repeated flooding from larger rainfall events are constructed at elevations much lower than the surrounding buildings, in some cases several feet. In addition to occurrences of flooding for large rainfall events, this area has had reports of flooding for relatively small rainfall frequencies as well. The results of this analysis do not reveal wide-spread flooding for the 2-yr rainfall event in this area. This could be attributed to the modification made to the land embankment or could be indicative of a secondary drainage issue which is beyond the scope and capability of this study. Based on these observations the structures along this stretch of the Resaca that are built at significantly lower elevations than surrounding structures may be viable candidates for the FEMA buy-out program if flooding persists. Additionally, because the partially dug-out land embankment is subject to erosion and the removed portion could fill back in subjecting the region to future drainage issues, it would also be favorable to have a permanent structure installed in place of the land embankment.

In addition to the floodplains displayed in Figures 3-9 & 3-10, the existing flood conditions from the 10-yr and 100-yr frequency events is summarized in Table 3-12 and Figure 3-11. The table lists an approximate description of the geographical location within the watershed, the corresponding river cross-section from the HEC-RAS model, the rainfall frequency, the contributing flow (Q) at the given cross-section and the corresponding water surface elevation (W.S.E.) in feet. Figure 3-11 depicts the profile of the computed water surfaces along the channel (Resaca) reach. Overall, the displayed water surface elevations were associated with floodplains covering approximately 12% of the entire land area for the 10-yr storm event and 18% for the 100-yr storm event. This level of flooding for each frequency storm is predicted to impact only 2 buildings for the 10-yr frequency rainfall event and 37 for the 100-yr. Table 3-11 displays the number of building predicted to fall within varying flood depth ranges for both rainfall events.

	Existing Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	1	15
0.5-1	1	9
1-1.5	0	3
1.5-2	0	5
2-2.5	0	4
2.5-3	0	1
3-3.5	0	0
3.5-4	0	0
4-4.5	0	0
4.5-5	0	0
5.0-8	0	0
8.0-12	0	0
>12.0	0	0
Total	2	37

Table 3-11. Number of Structures/Buildings predicted to be flooded in the LRRV watershed area under existing conditions for the 10-yr and 100-yr frequency rainfall

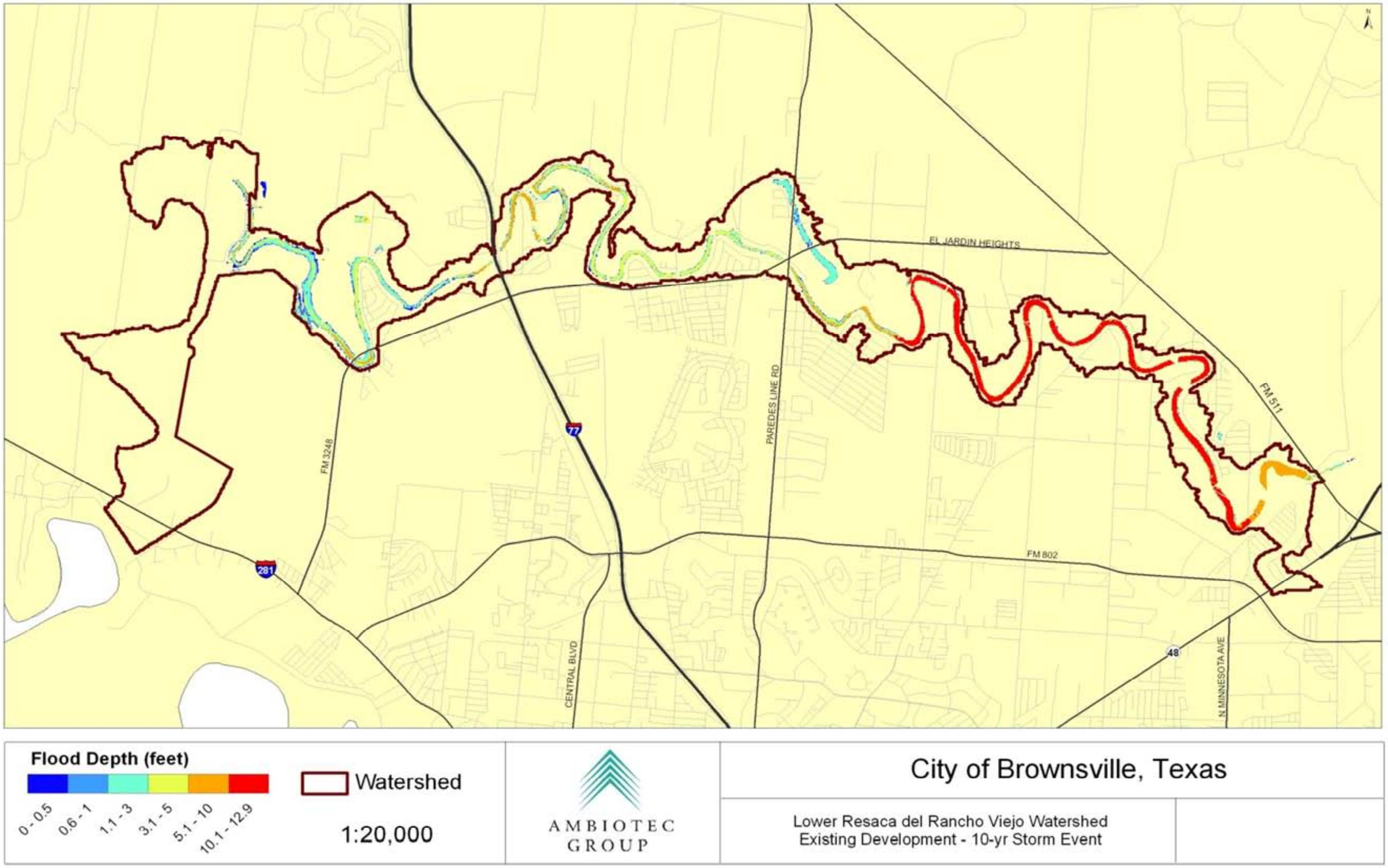

Figure 3-9. Existing development floodplain for the LRRV watersheds assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

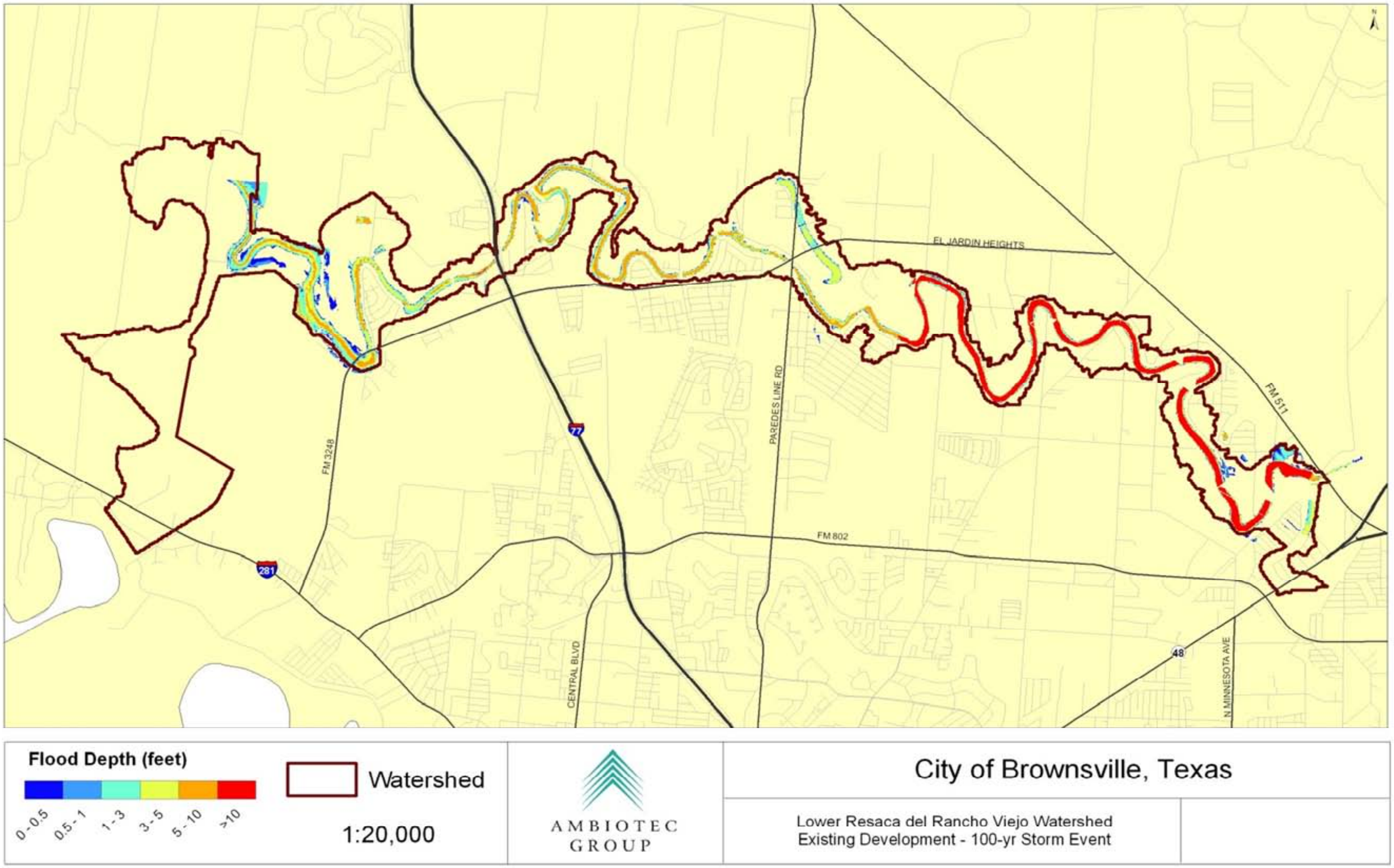

Figure 3-10. Existing development floodplain for the LRRV watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Furthest Upstream Area	99344.57	10-yr	143.9	29.8
	99344.57	100-yr	230.7	32.11
Alton Gloor (FM 3248)	86422.92	10-yr	204.7	29.48
	86422.92	100-yr	364	32.02
Lakeway Drive	81775.41	10-yr	196.9	26.19
	81775.41	100-yr	359.7	27.9
Downstream of US 77/83	74557.76	10-yr	106.6	23.74
	74557.76	100-yr	165.5	24.93
Dennett Rd./Stillman Rd.	62008.83	10-yr	199.9	22.96
	62008.83	100-yr	367	24.71
Stagecoach Trail	53542.97	10-yr	188.3	21.28
	53542.97	100-yr	343	23.06
Paredes Line Rd.	47681.73	10-yr	181.5	21.23
	47681.73	100-yr	351	22.94
Dana Rd.	36180.49	10-yr	75.3	20.43
	36180.49	100-yr	312.7	22.4
Robindale Rd.	25334.66	10-yr	113.6	19.11
	25334.66	100-yr	250.3	22.39
Old Port Isabel Rd.	21421.27	10-yr	115.6	19.11
	21421.27	100-yr	185.1	22.39
NW of Morrison Rd. and Salida del Sol	13058.5	10-yr	115.6	19.11
	13058.5	100-yr	185.1	22.39
Charmaine Rd.	6957.445	10-yr	161	16.93
	6957.445	100-yr	258.9	22.39
Near Railroad Crossing before Ditch	3144.294	10-yr	161	16.93
	3144.294	100-yr	258.9	22.39
Outfall (Ditch)	298.6842	10-yr	87.9	4.63
	298.6842	100-yr	138.7	4.96

Table 3-12. Lower RRV Existing Development Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

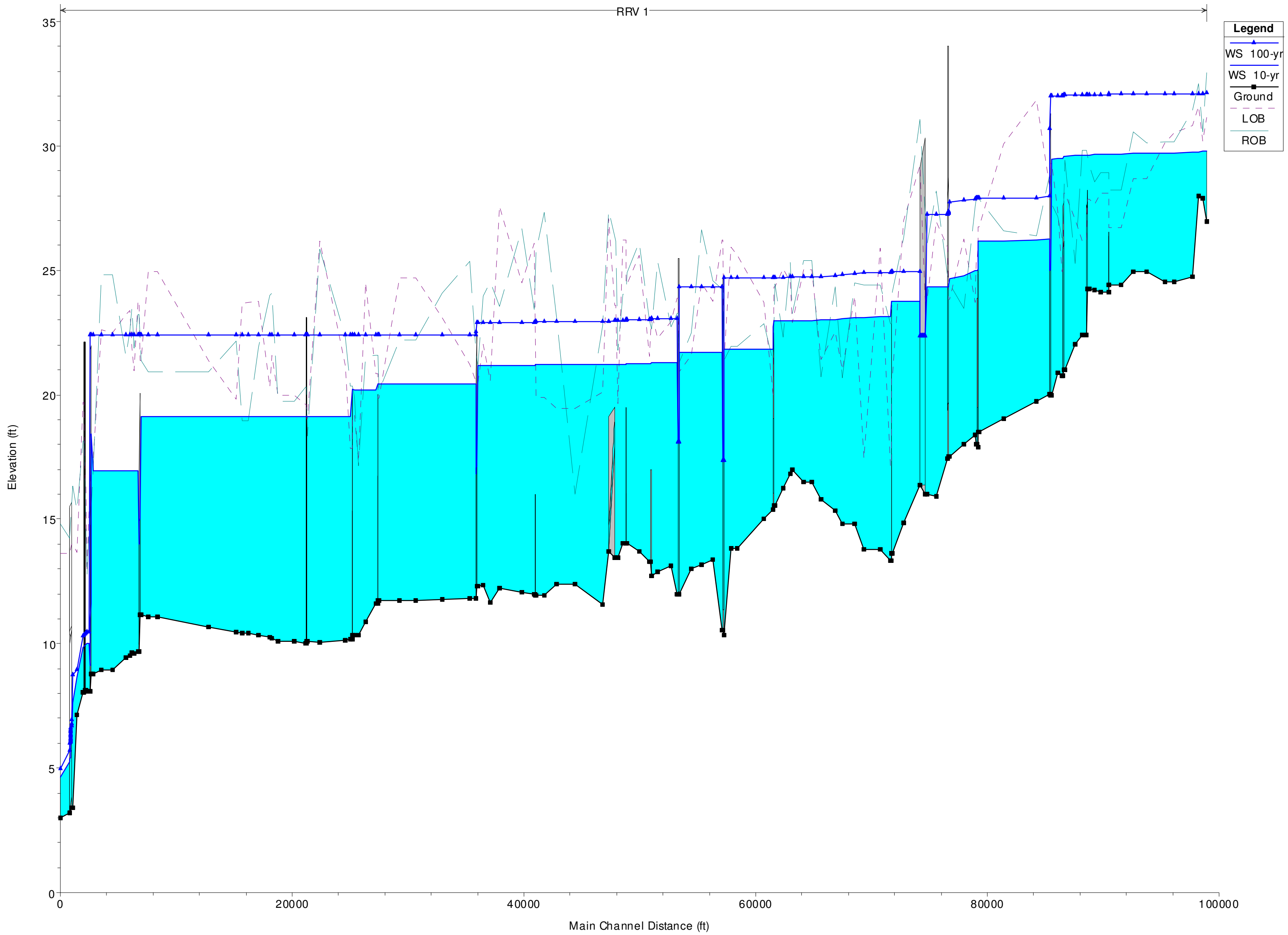

Figure 3-11. Water surface elevation profile for LRRV under existing conditions

3.2.3 Lower RRV Full Development Analysis

This section discusses the impact that allowing the watershed to become fully developed without implementing runoff control measures would have on water surface profiles and total floodplain area. To reiterate the importance of maintaining existing development peak flow rates the 10-yr and 100-yr floodplains for LRRV under un-regulated full development conditions along with the computed water surface elevations at various points along the Resaca are displayed in Figures 3-12 & 3-13 and Table 3-14 on the following pages. Profiles of computed water surface elevations for both the 10-yr and 100-yr storm may be viewed in Figure 3-14. The implementation of the full development scenario on LRRV resulted in a floodplain covering approximately 15% of the entire watershed land area (614 acres) for the 10-yr rainfall event and nearly 20% for the 100-yr event (768 acres). The result is the inundation of approximately 14 buildings for a 10-yr rainfall event with assumed constant area-wide rainfall distribution and 78 buildings for the 100-yr event. Table 3-13 summarizes the number of buildings predicted to be impacted by varying water depths for both the 10-yr and 100-yr events. In addition to the increase of floodplain area, water surface elevations increased as a result of the full development scenario from anywhere to 0.1-ft to over 2-ft for either rainfall frequency event.

	Full Dev. Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	3	36
0.5-1	1	17
1-1.5	5	10
1.5-2	4	6
2-2.5	1	6
2.5-3	0	2
3-3.5	0	1
3.5-4	0	0
4-4.5	0	0
4.5-5	0	0
5.0-8	0	0
8.0-12	0	0
>12.0	0	0
Total	14	78

Table 3-13. Number of Structures/Buildings predicted to be flooded in the LRRV watershed area under assumed full development conditions for the 10-yr and 100-yr frequency rainfall

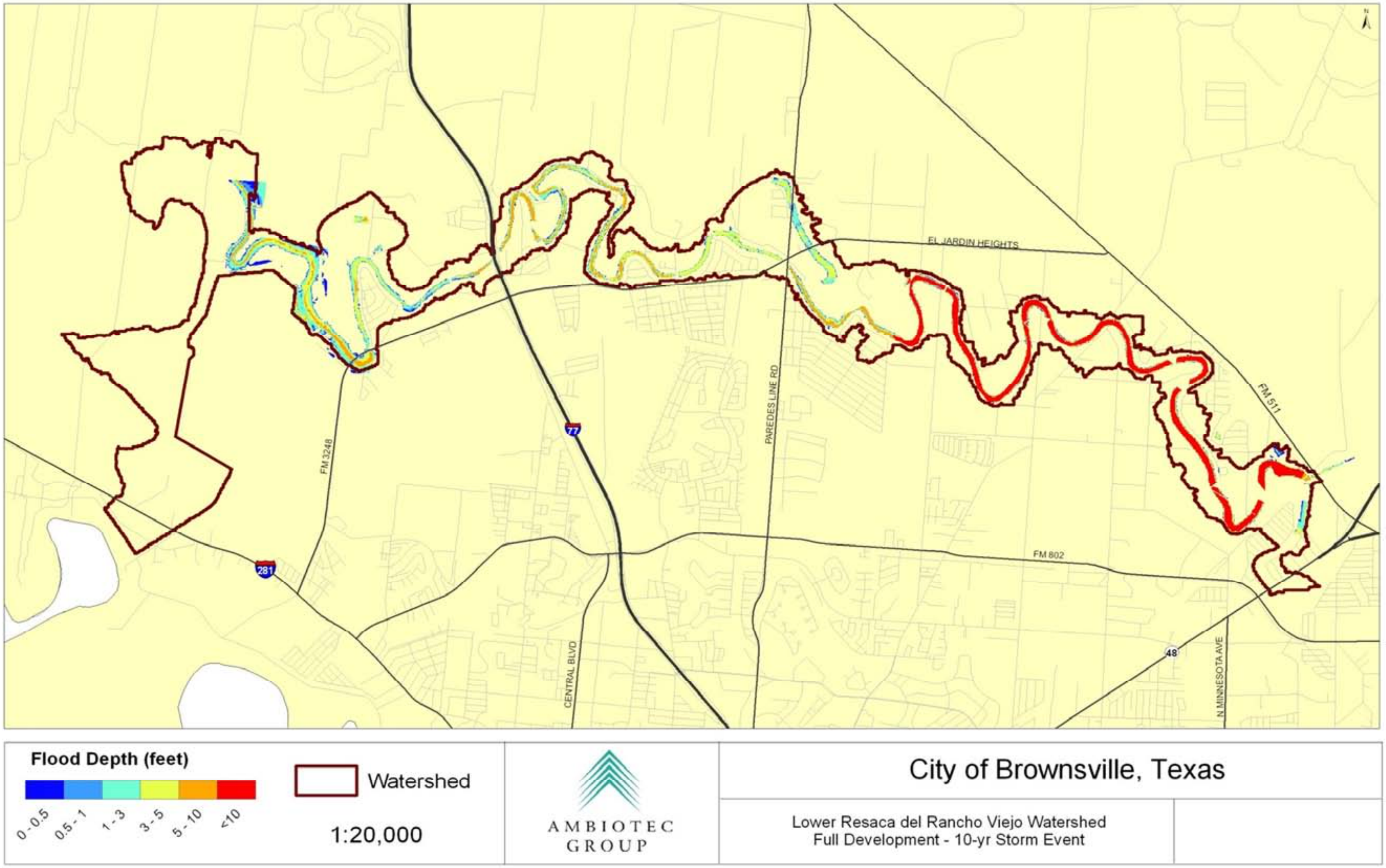

Figure 3-12. Full development floodplain for the LRRV watersheds assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

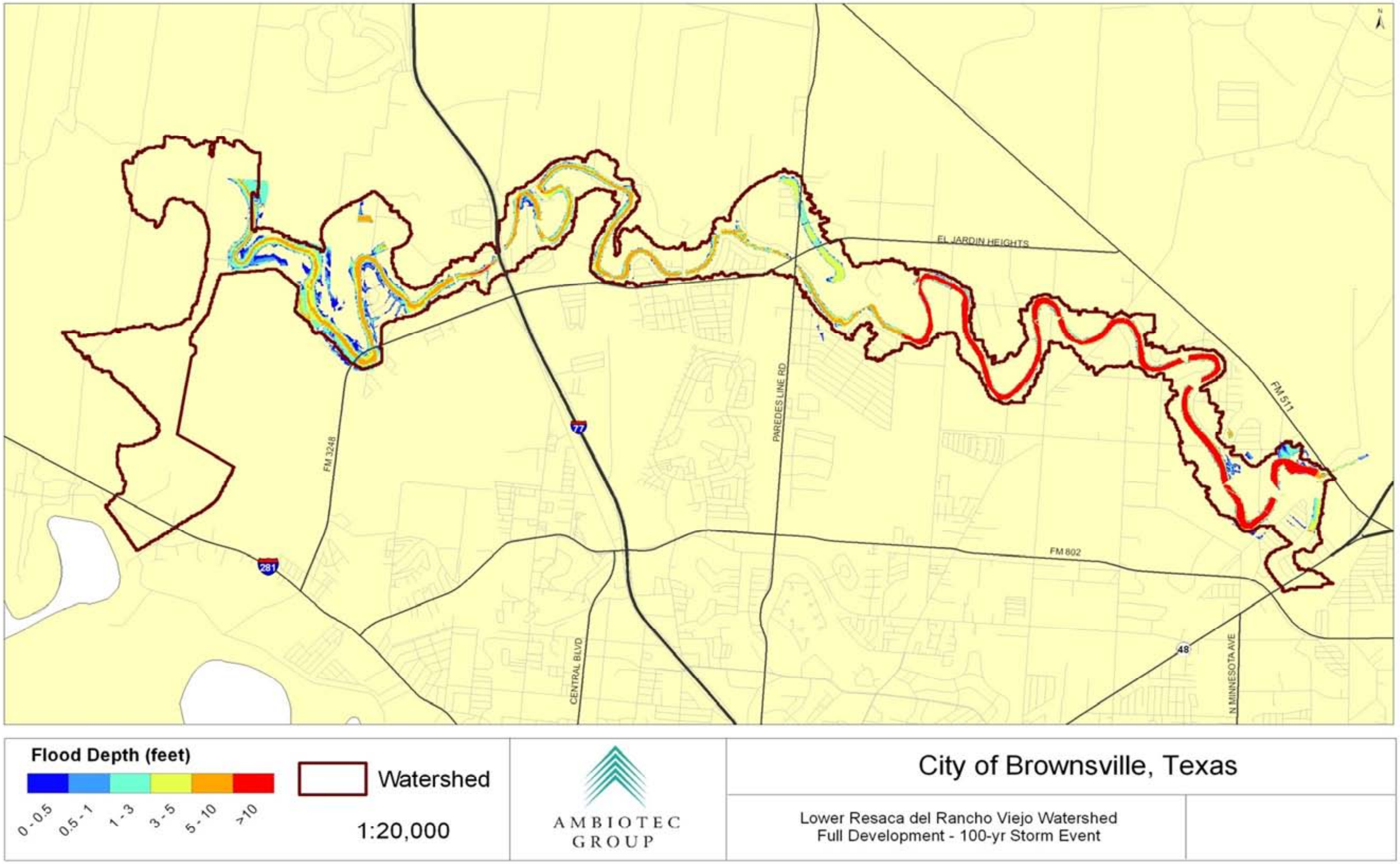

Figure 3-13. Full development floodplain for the LRRV watersheds assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Furthest Upstream Area	99344.57	10-yr	179.9	31.62
	99344.57	100-yr	288.4	32.34
Alton Gloor (FM 3248)	86422.92	10-yr	255.9	31.52
	86422.92	100-yr	455	32.2
Lakeway Drive	81775.41	10-yr	246.1	26.41
	81775.41	100-yr	449.6	29.97
Downstream of US 77/83	74557.76	10-yr	133.3	23.93
	74557.76	100-yr	206.9	25.58
Dennett Rd./Stillman Rd.	62008.83	10-yr	249.9	23.19
	62008.83	100-yr	458.8	25.34
Stagecoach Trail	53542.97	10-yr	235.4	22.26
	53542.97	100-yr	428.8	23.17
Paredes Line Rd.	47681.73	10-yr	226.9	22.22
	47681.73	100-yr	438.8	23.06
Dana Rd.	36180.49	10-yr	94.1	21.05
	36180.49	100-yr	390.9	22.53
Robindale Rd.	25334.66	10-yr	142	21.04
	25334.66	100-yr	312.9	22.49
Old Port Isabel Rd.	21421.27	10-yr	144.5	21.04
	21421.27	100-yr	231.4	22.49
NW of Morrison Rd. and Salida del Sol	13058.5	10-yr	144.5	21.03
	13058.5	100-yr	231.4	22.49
Charmaine Rd.	6957.445	10-yr	201.3	21.03
	6957.445	100-yr	323.6	22.48
Near Railroad Crossing before Ditch	3144.294	10-yr	201.3	21.03
	3144.294	100-yr	323.6	22.48
Outfall (Ditch)	298.6842	10-yr	109.9	4.78
	298.6842	100-yr	173.4	5.15

Table 3-14. Lower RRV Full Development Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

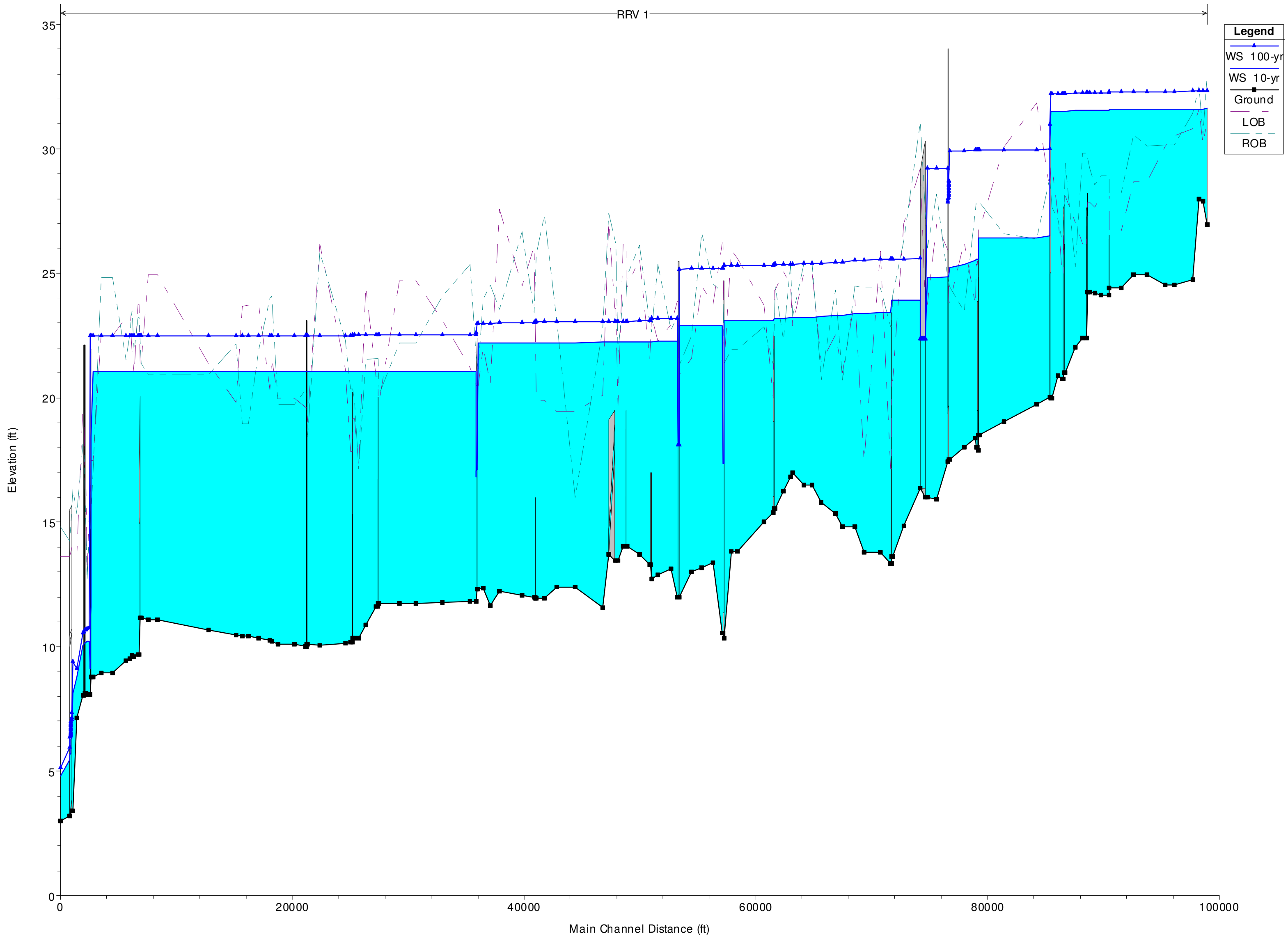

Figure 3-14. Water surface elevation profile for LRRV under full development conditions

4.0 Merged Model for RDLG, NMD, and TR Watersheds

As discussed in Section 1.3 #'s 4-5, the hydrologic models developed in the 2006 Flood Study for RDLG and TR were re-created in HEC-HMS format for consistency with the rest of the planning area. The HEC-RAS models for these two watersheds, which both have outfalls at NMD, were then merged with the NMD HEC-RAS model to more accurately simulate the backwater effects that water levels in NMD have on both RDLG and TR. Furthermore, this model linkage allowed for a more accurate representation of stormwater transfers between the different watersheds. The development of each HEC-HMS model (RDLG and TR) will briefly be discussed in section 4.1. Section 4.2 will discuss the development of the linked HEC-RAS model and the overall results of this analysis under the existing development scenario. Due to the fact that the existing conditions of this combined watershed area are almost fully developed, the full development scenario was not analyzed.

4.1 Conversion of RDLG and TR Models to HEC-HMS Format

4.1.1 Description of RDLG Watershed and Development of HEC-HMS Model

The RDLG watershed is located south of the CCDD1 watershed and north of the NMD watershed. The Resaca flows from a northwest to southeast direction and is approximately 17.3 river miles long. The outfall is located where it meets NMD near the intersection of Southmost Rd. and Morningside Rd. where there is an overflow structure carrying water from the Resaca into the drainage ditch. In recent years, the Brownsville Irrigation District installed a pump near the outfall to enable them to pump water levels in the Resaca down in anticipation of large rainfall events. The pump has the capacity to remove approximately 53 cfs and the pumped water flows into a pair of reinforced concrete pipes on the opposite side of Southmost Rd. that eventually terminate to the Rio Grande River.

The watershed is approximately 2,944 acres (4.6 square miles) in area and has average slopes between 0.6% and 1.7%. The watershed boundary was delineated in the 2006 study and broken down into 18 subareas using HEC-GeoHMS software as described in the 2006 study report (Figure 4-1). The boundaries previously delineated were again used in this analysis in the HEC-HMS model. The majority of the watershed is developed and consists mostly of residential type landuse although there are some commercial type areas as well. The TC&R values by subwatershed used for the development of the HEC-HMS model are displayed in Table 4-1. Like the other planning area models the Clark Method was used along with the modified puls storage routing method. To best capture the behavior of individual Resaca "pools" throughout the system, a reach was assigned to each Resaca segment broken up by a culvert, weir, or some other type of manmade or natural barrier to flow. The flows computed from this model were entered into the linked HEC-RAS model (discussed in Section 4.2) and used to develop floodplains and compute expected water depths for given rainfall frequency events.

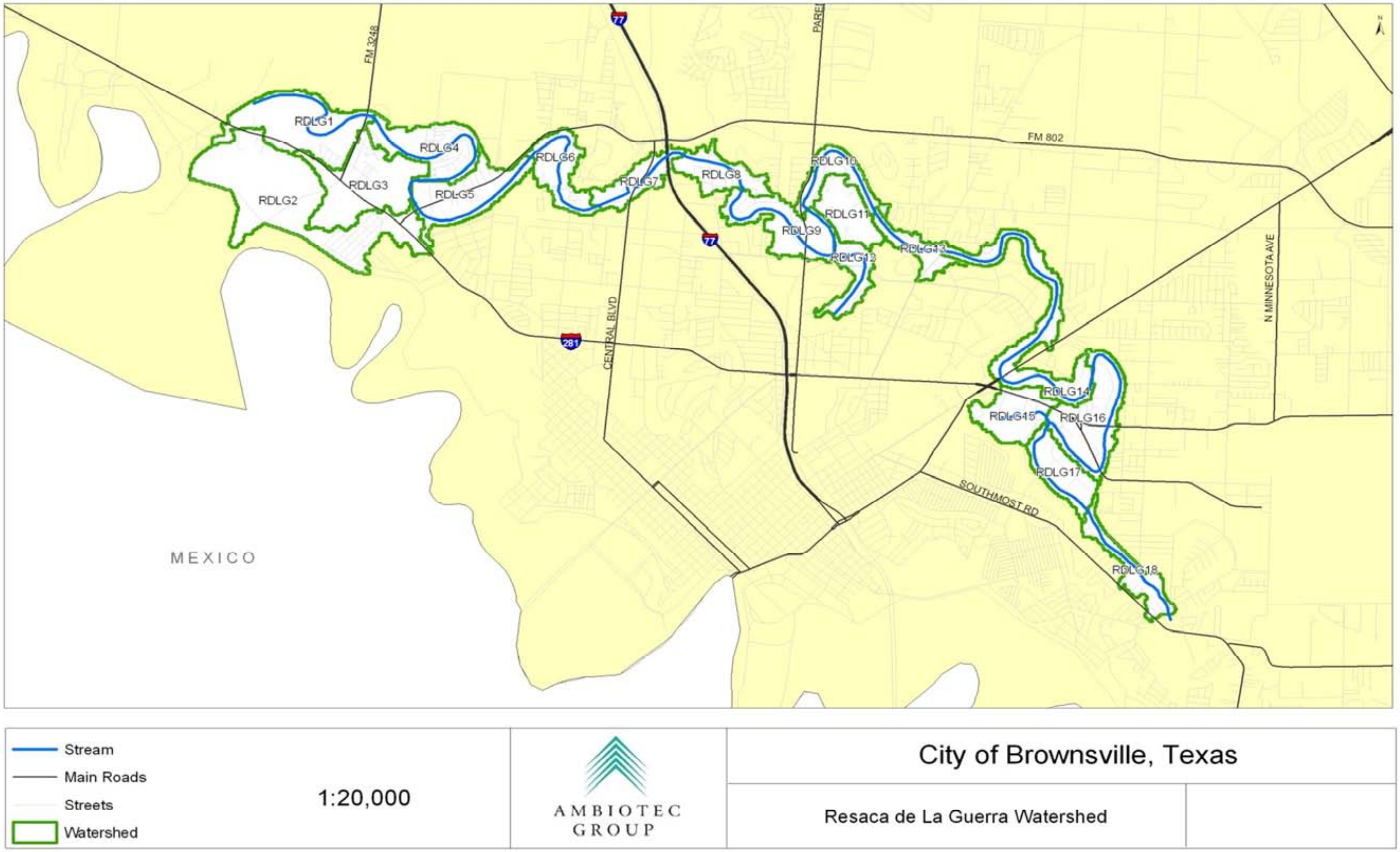

Figure 4-1. Subwatershed map for RDLG – Defined in 2006 Flood Protection Plan

Subareas	Drainage Area (Acres)	TC (Hours)	R (Hours)
RDLG1	254.3	1.16	7.6
RDLG2	489.7	2.09	4.17
RDLG3	249.9	0.65	3.59
RDLG4	174	2.62	6.15
RDLG5	180.8	4.63	8.43
RDLG6	122.3	0.57	3
RDLG7	89.9	1.07	2.31
RDLG8	101.7	0.29	2.4
RDLG9	169.9	1.08	2.05
RDLG10	93.6	1.16	7.42
RDLG11	109.7	0.41	1.92
RDLG12	78.4	0.43	1.94
RDLG13	68.5	0.13	1.46
RDLG14	237.4	1.8	4.03
RDLG15	125.5	0.24	2.03
RDLG16	209.4	5.62	6.75
RDLG17	115.5	0.67	2.77
RDLG18	96.7	6.19	10.55

Table 4-1. TC&R Values for the RDLG Watershed

4.1.2 Description of TR Watershed and Development of HEC-HMS Model

The TR watershed is located south of the NMD watershed and incorporates the majority of the downtown area. The Resaca generally flows from a northwest to southeast direction and is approximately 7.75 river miles long. The outfall is located where it meets NMD approximately 1.4 miles upstream of where RDLG terminates at NMD. The last segment of the TR system is a ditch and runs openly into the NMD. Just upstream of the outfall is the Impala Pump Station with an approximately 540 cfs capacity when all pumps are running. The pump station pumps excess water from the TR Ditch/NMD Ditch to the Rio Grande over the levee.

The watershed is approximately 3,648 acres (5.7 square miles) in area and has average slopes between 0.4% and 1.6%. The watershed boundary was delineated in the 2006 study and broken down into 35 subareas using HEC-GeoHMS software as described in the 2006 study report (Figure 4-2). The boundaries previously delineated were again used in this analysis in the HEC-HMS model. The majority of the watershed is developed and consists mostly of high intensity residential and commercial/industrial type landuse. The TC&R values by subwatershed used for the development of the HEC-HMS model are displayed in Table 4-2. Again the Clark Method was used for unit hydrograph computations and for storage routing the modified puls method was used. The flows computed from this model were entered into the linked HEC-RAS model (discussed in Section 4.2) and used to develop floodplains and compute expected water depths for given rainfall frequency events.

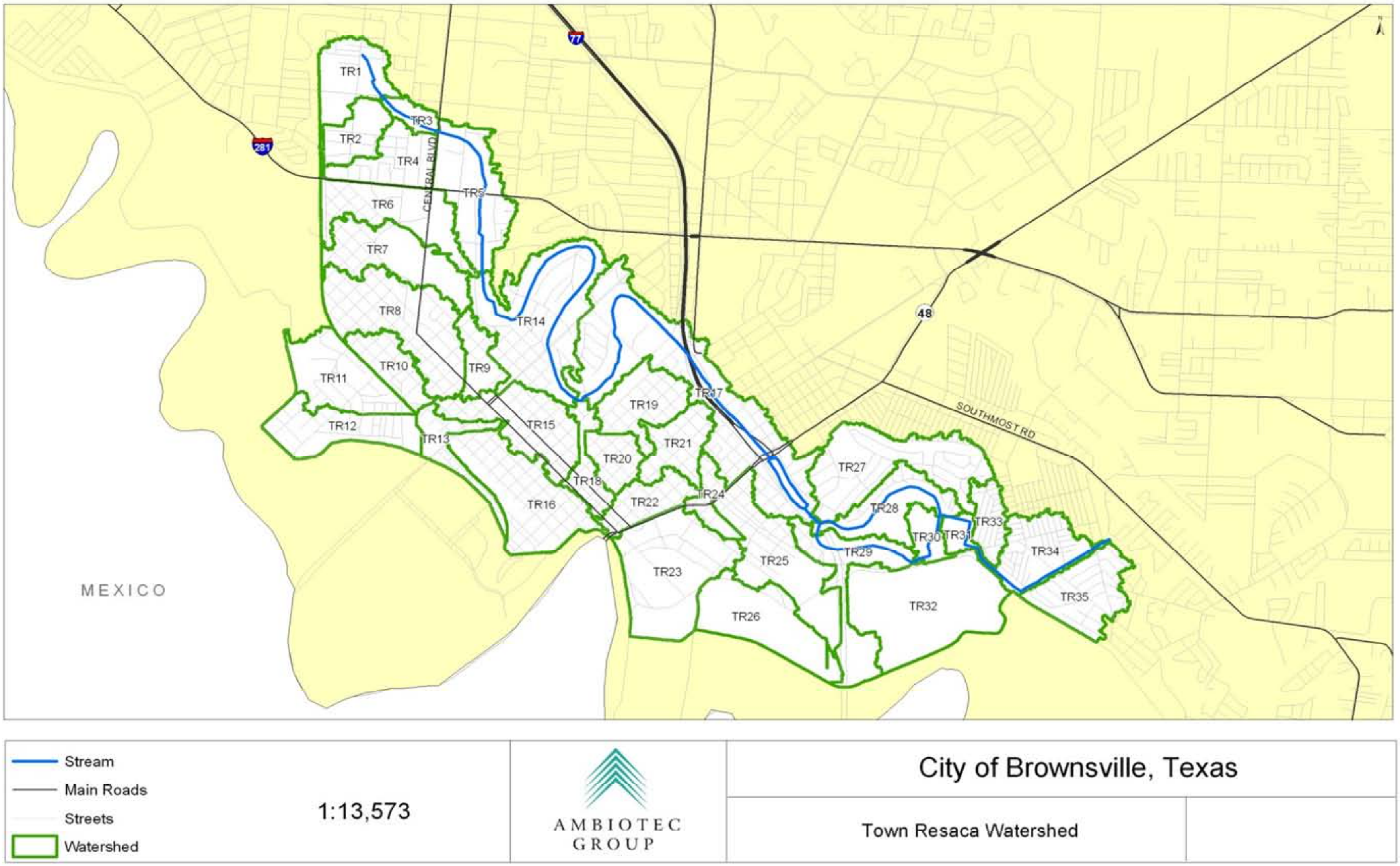

Figure 4-2. Subwatershed map for TR - Defined in 2006 Flood Protection Plan

Subareas	Drainage Area (Acres)	TC (Hours)	R (Hours)
TR1	84.45	0.05	2.48
TR2	53	0.24	1.88
TR3	21.92	0.39	1.71
TR4	75.72	0.31	1.72
TR5	133.84	0.38	2.16
TR6	116.51	0.46	2.45
TR7	117.6	0.22	2.17
TR8	177.94	0.59	2.22
TR9	43.94	0.16	1.26
TR10	56.35	0.25	1.59
TR11	109	0.34	2.18
TR12	99.98	0.29	2.03
TR13	51.12	0.72	2.03
TR14	231.65	0.5	3.73
TR15	119.24	0.34	1.78
TR16	157.11	0.45	2.11
TR17	345.87	0.59	3.12
TR18	42.92	0.32	1.65
TR19	85.57	0.35	1.75
TR20	44.41	0.17	1.2
TR21	66.23	0.22	1.57
TR22	57.53	0.34	1.6
TR23	185.69	0.72	2.33
TR24	33.39	0.21	1.3
TR25	149.82	0.6	2.49
TR26	132.65	0.76	3.45
TR27	173.36	0.29	2.78
TR28	91.47	0.63	2.54
TR29	107.58	2.62	4.28
TR30	30.41	0.29	2.97
TR31	21.03	0.14	1.32
TR32	224.56	0.14	1.89
TR33	40.64	0.18	1.48
TR34	90.93	0.28	1.75
TR35	99.39	0.27	1.66

Table 4-2. TC&R values for the Town Resaca Watershed

4.2 Development of Linked HEC-RAS Model and Results of Existing Analysis

A single HEC-RAS model was developed for the inter-connected three watershed area consisting of RDLG, NMD, and TR (Figure 4-3). To create this model the HEC-GeoRAS extension was used to develop a RAS import file based on the streamlines and cross sections defined in ArcMap and the LIDAR derived topographic data. The data used for the import file was developed during the 2006 study but merged to create a single import file with cross-sections and streamlines for all three watershed areas. The landuse data that is used to derive Manning's roughness coefficients was updated from the previous study with the more current data described in Section 2.1. The entire combined watershed area is approximately 12,800 acres (20 square miles) with average slopes of less than 1%. The watershed is approximately 86% developed with varying levels of residential development along with commercial/industrial type landuse. Because the combined watershed area is close to full development levels, only the existing condition scenario was analyzed.

Overall, the HEC-RAS model for the merged RDLG, NMD, TR model consisted of nearly 400 cross-sections with the same geometry and topographic data as the cross-sections used in the individual models from the 2006 study. However, there were many factors that have changed since the 2006 study models were developed that were reflected in this analysis. There were several culverts that were modified since the development of the 2006 models (culverts included in the model may be viewed in Tables 4-3 through 4-5) in addition to the pump that was installed at the outfall of RDLG as discussed in Section 4.1.1. Additionally, landuse in the region has changed since the previous analysis. Finally, since the three watersheds have been merged to represent the inter-connectedness of the three drainage systems, this model is now able to simulate the backwater effects that result from varying water surface elevations in the other two systems which is an improvement over the simulation completed in the 2006 analysis. The combined impact of these factors changes both the overland and channel flow characteristics across the tri-watershed area resulting in minor to moderate differences in predicted flows. Changes in predicted flows are further compounded as a result of the difference in the timing of the overland flow contributions to the Resaca networks due to the use of the lumped parameter (HEC-HMS) modeling software for this study as compared to the distributed type model (VFLO) that was used in 2006.

The computed water surface elevations (WSE's) for both the RDLG and TR watersheds showed some deviation from those WSE's computed in the 2006 study especially for the smaller frequency storms but the overall floodplain area changed very little since the increased flows were still within the Resaca storage capacity to accommodate. In the RDLG watershed computed WSE's were elevated in some sections of the Resaca and lowered in other portions. This is largely attributed to changes in overland peak flows as a result of increased development along with several culvert improvements that were implemented since the previous analysis. In the TR watershed a general rising of computed water surface elevations was observed, especially for the larger storm events and is largely attributed to the backwater effect of raised WSE's in the NMD. Computed WSE's in the NMD watershed remained relatively constant to that of the previous study until the segments further downstream (east of Southmost Rd.) after the RDLG inflow. This reduction of WSE's is believed to be largely attributed to the

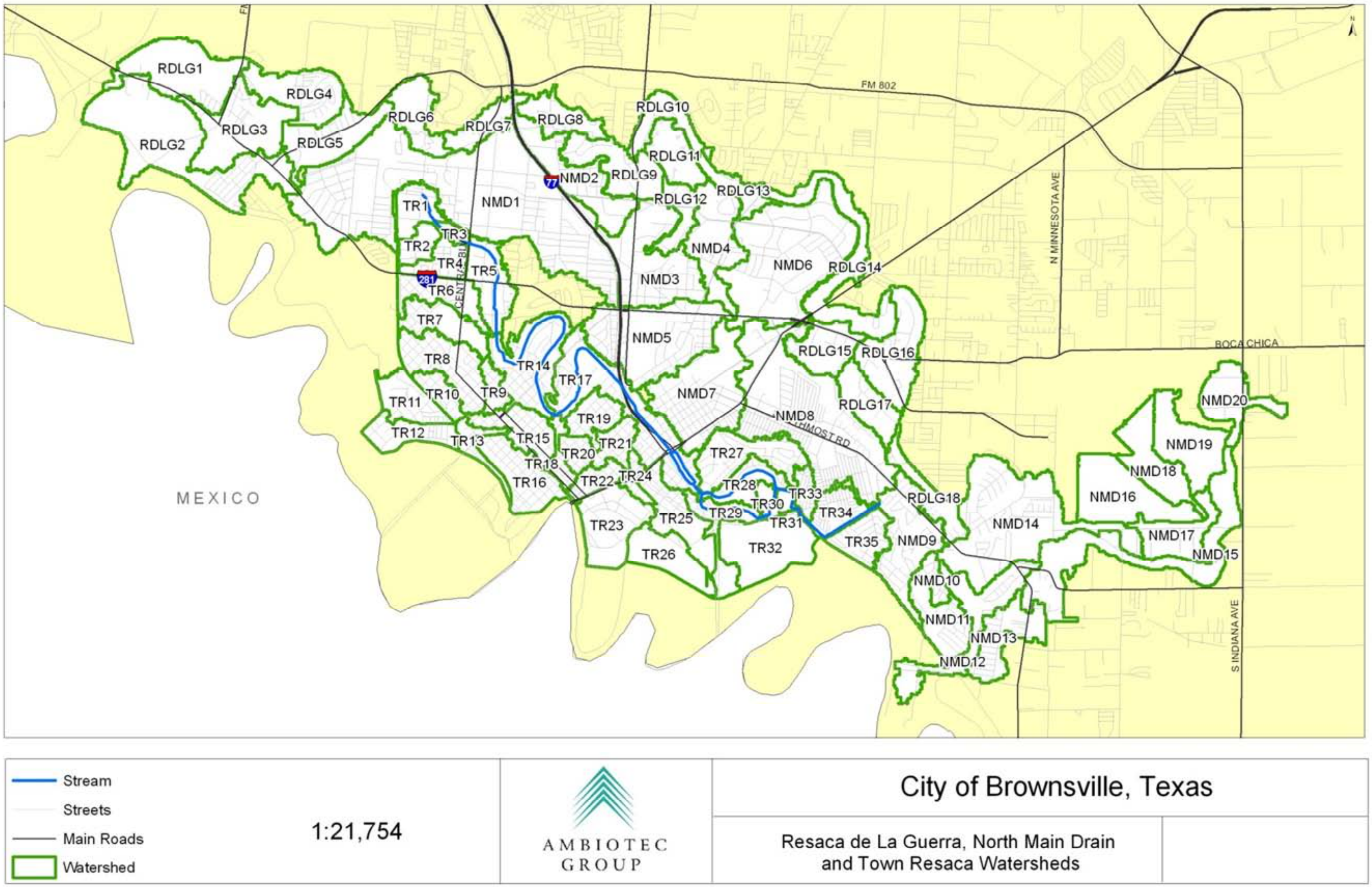

Figure 4-3. Three watershed area with subwatersheds included in the merged HEC-RAS analysis

Street Name	Station	Type	Number	Size
Alton Gloor	84300	RCP	1	3-ft
Laredo (1st Crossing)	80900	Box	1	8-ft x 4-ft
Laredo (2nd Crossing) - Overflow	76160	Box	2	6-ft x 2-ft
FM 802	74000	RCP	2	4-ft
Laredo - Overflow	73600	Box/RCP	2	2, 6-ft x 4-ft, 2, 4-ft
Mercedes Rd	70200	Weir	1	N/A
Railroad	68000	Bridge	1	N/A
Golf Course	66600	RCP	2	1.5-ft
Golf Course	65600	Bridge	1	N/A
Golf Course	64000	RCP	1	2-ft
Golf Course	63300	RCP	2	2-ft
Golf Course	63000	RCP	1	1.5-ft
Golf Course	62840	RCP	1	2-ft
Golf Cart Bridge	61200	Bridge	1	N/A
Golf Course/Old Hwy 77	60500	Bridge	1	N/A
Old Highway 77	60200	Bridge	1	N/A
Central Blvd	60000	RCP	1	4-ft
US 83/77	58600	Box	1	5-ft x 5-ft
N/A	57500	Weir	1	N/A
Old Alice	56500	RCP	2	4.33-ft
Hidden Valley Drive	53500	RCP	2	2-ft
Hike and Bike Bridge	50900	Bridge	1	N/A
N/A	50400	Weir	1	N/A
Paredes Line	49800	RCP	1	4.33-ft
Palo Verde Drive	46000	RCP	1	3.5-ft
N/A	37000	Weir	1	N/A
Old Port Isabel	36400	Box	2	8-ft x 8-ft
N/A	28500	Bridge	1	N/A
Price Rd	27500	Box	1	10-ft x 8-ft
Padre Island Highway	25490	Weir/RCP	1	N/A/5.83-ft
Boca Chica	15600	Box	2	10-ft x 8-ft
Boca Chica	15543	weir	1	N/A
Billy Mitchell Blvd	13600	RCP	3	3.5-ft
Acacia Lake Drive	11100	Bridge	1	N/A
Morningside Rd	4700	RCP	3	1, 1.25-ft and 2, 2.5-ft
Morningside Rd	500	RCP	3	2.5-ft
N/A - Overflow	60	Box	1	6' x 6'
Shidler	3800	RCP	1	2-ft
Price Rd	1700	RCP	1	2-ft
Eagle	300	RCP	1	2-ft
Owens Rd	800	Bridge	1	N/A

Table 4-3. Culverts in RDLG

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

Table 4-4. Culverts in NMD

Street Name	Station	Type	Number	Size
Los Ebanos Blvd.	39450	RCP	1	1.5-ft
N/A	39089	Weir	1	N/A
Central Blvd.	37600	RCP	1	1.5-ft
Coria St.	36800	RCP	1	1.25-ft
Boca Chica Blvd.	35150	RCP	1	2-ft
Belthair St.	33700	RCP	1	1.5-ft
N/A	31788	Weir	1	N/A
Calle Retama	30300	Box	1	10-ft x 8-ft
Pedestrian Bridge	28100	Box	1	12-ft x 6.5-ft
Ringgold St.	26400	Box	1	10-ft x 10-ft
Calle Retama	25500	Box	1	10-ft x 8-ft
N/A	25100	Box	1	9-ft x 4-ft
Palm Blvd.	24700	Box	1	10-ft x 6-ft
Palm Blvd	24450	Weir	1	N/A
Old Alice	22600	Box	2	9-ft x 4-ft
Railroad Crossing	22100	Box	3	8-ft x 10-ft
Ringgold St.	20300	Weir	1	N/A
6th St.	17200	Box	2	9-ft x 9-ft
7th	17111	Box	2	10-ft x 8-ft
US 83/77	16756	Box	2	10-ft x 8-ft
Railroad Crossing	15800	Box	2	10-ft x 8-ft
12th St.	15400	Box	2	10-ft x 9-ft
13th St.	14000	Box	2	10-ft x 9-ft
14th St.	13600	Box	2	10-ft x 9-ft
International Blvd	12285	Box	2	10-ft x 9-ft
Father Ballard	11000	Bridge	1	N/A
Weir	10800	weir	1	N/A
US 83/77	10400	Bridge/Weir	1,1	N/A
WWTP Facility Crossing	4700	2 RCP, 1 CMP	3	2, 36-ft 1, 6.67-ft
East Ave.	4500	Bridge	1	N/A
US 83/77	3800	Bridge	1	N/A
Impala Drive	2600	Bridge	1	N/A
Calle Milpa Verde Dr.	1700	Bridge	1	N/A
Tulipan	400	Bridge	1	N/A

Table 4-5. Culverts in TR

decreased flow rates coming into the ditch from RDLG because of the pump that was installed at the outfall of RDLG. Once again though, the overall impact to the entire floodplain area was minimal.

In all, the overall floodplain area for the combined three watershed area in the 2006 study was approximately 3,040 acres for the 10-yr event and 3,963 for the 100-yr event. During this analysis the combined floodplain area for each event was calculated to be approximately 3,072 acres and 4,224 acres respectively which represent approximately a 1.1% and 6.6% increase in floodplain area over that of the previous analysis. The computed floodplains from this analysis for both the 10-yr and 100-yr storm event may be viewed in Figures 4-4 & 4-5. These computed floodplains inundate approximately 24% and 33% of the entire watershed area for the 10-yr and 100-yr storm respectively and result in the flooding of approximately 1790 and 4169 buildings for each of the two events during a single occurrence. A summary of the expected number of buildings flooded within a given depth range as a result of a 10-yr and 100-yr frequency rainfall event is displayed in Table 4-6. The 2-yr estimates for flooded buildings are also being presented for comparison purposes to proposed options that are discussed later in Sections 6.1 and 6.2. Computed WSE's may be viewed in Tables 4-7 through 4-9 and schematics of the WSE profiles for each watershed are displayed in Figures 4-6 through 4-8.

It may be observed that while the schematics for both of the Resaca networks show WSE's that are commonly above the bank elevations, it should be noted that for the Resacas there are generally lower banks near the edge of the Resaca and an upper bank a distance away from the edge where the majority of development begins. Therefore, there can be “flooding”in the resacas that technically comes past the lower bank but would still have to rise in some cases several feet before impacting constructed developments that sit at the higher bank.

	Existing Conditions		
Water Depth (ft)	**2-Yr**	**10-Yr**	**100-Yr**
0-0.5	369	666	1377
0.5-1	220	442	971
1-1.5	100	328	694
1.5-2	39	189	491
2-2.5	18	75	306
2.5-3	6	35	156
3-3.5	10	11	69
3.5-4	2	8	40
4-4.5	1	10	21
4.5-5	2	9	11
5.0-8	3	16	31
8.0-12	0	1	2
>12.0	0	0	0
Total	770	1790	4169

Table 4-6. Number of Structures/Buildings predicted to be flooded in the merged watershed model (RDLG, NMD, and TR) under existing conditions for the 2-yr, 10-yr and 100-yr frequency rainfall

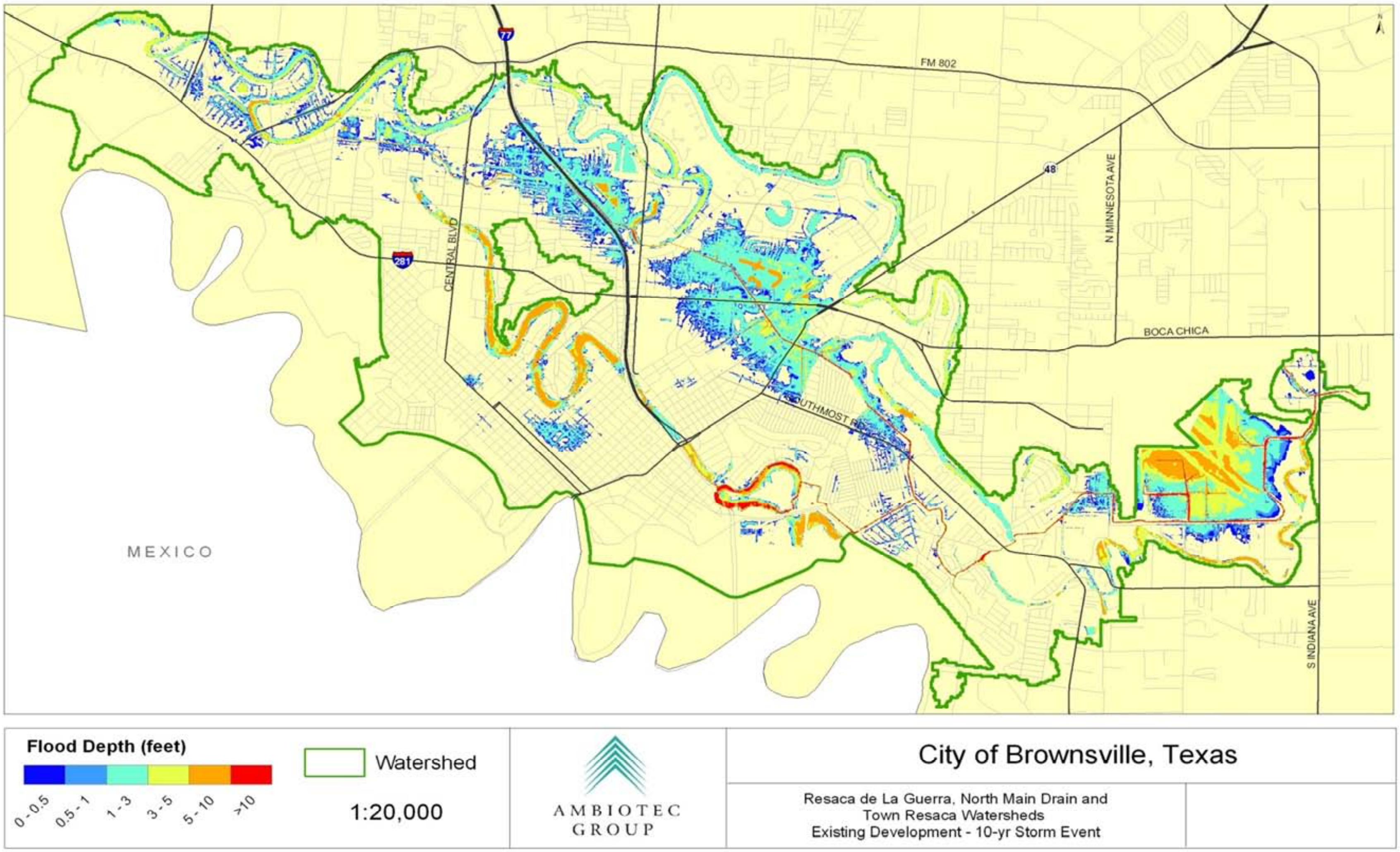

Figure 4-4. Existing development floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

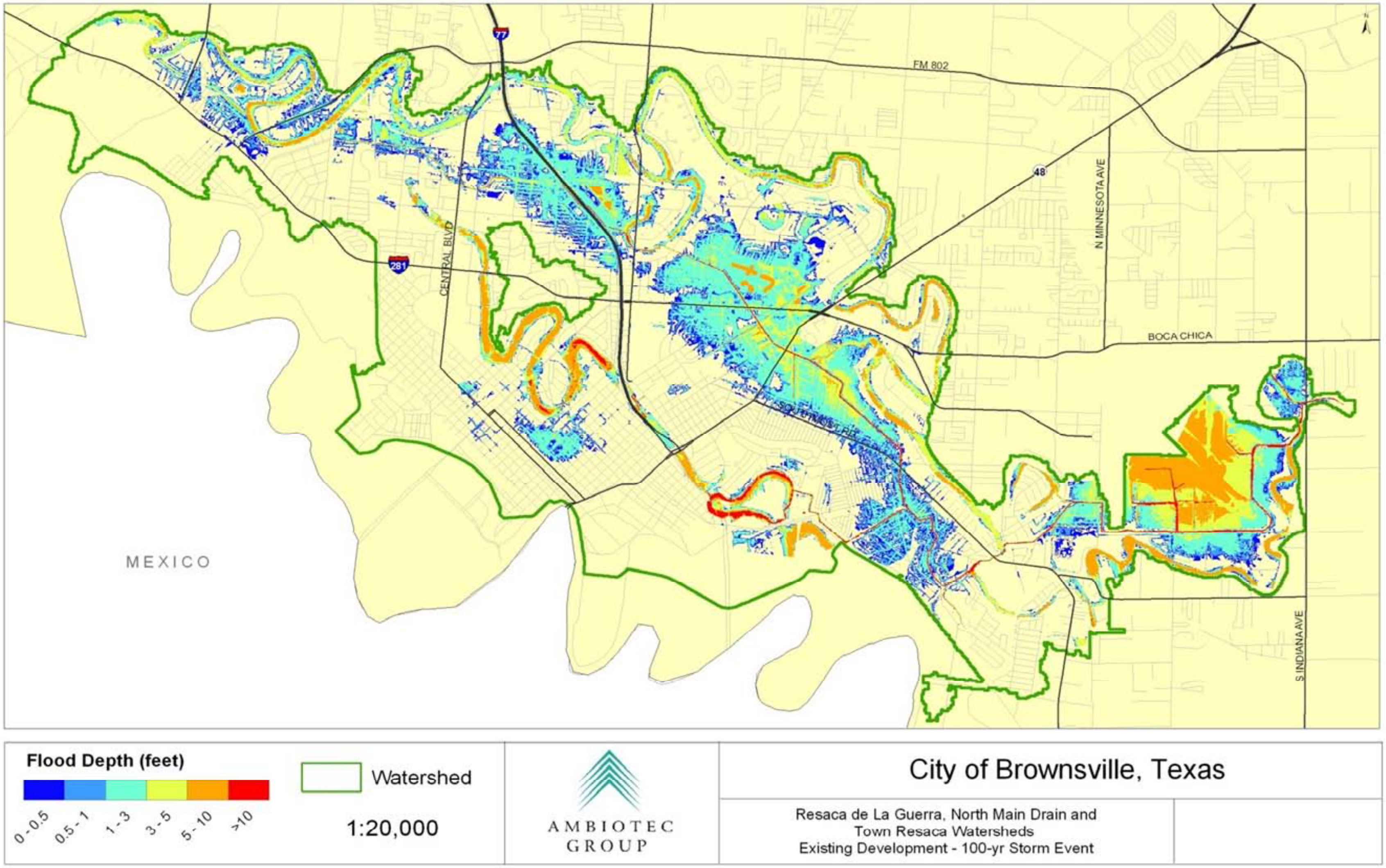

Figure 4-5. Existing development floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Alton Gloor	84480.30	10-yr	3.8	34.04
	84480.30	100-yr	18.3	34.42
Laredo Rd.	75741.20	10-yr	105.1	34.04
	75741.20	100-yr	162.5	34.42
Golf Course (upstream VICC)	67843.07	10-yr	654.9	33.24
	67843.07	100-yr	999.9	33.92
Central Blvd.	59913.97	10-yr	575	31.93
	59913.97	100-yr	1012.7	32.53
US 77/83	58006.56	10-yr	575	31.51
	58006.56	100-yr	1012.7	32.05
Old Alice	56431.98	10-yr	528.9	30.41
	56431.98	100-yr	1041.5	31.08
Paredes Line Rd.	49414.23	10-yr	421.1	29
	49414.23	100-yr	1024.5	30.12
Old Port Isabel	36395.52	10-yr	287.1	26.19
	36395.52	100-yr	956.3	28.59
Price Rd.	27465.97	10-yr	287.1	25.57
	27465.97	100-yr	956.3	27.07
Hwy 48	24796.52	10-yr	242.5	23.66
	24796.52	100-yr	856.5	26.06
Boca Chica	15543.40	10-yr	242.5	23.33
	15543.40	100-yr	856.5	24.9
Billy Mitchell Blvd.	13591.11	10-yr	237.4	22.36
	13591.11	100-yr	812.6	24.69
Acacia Lake Drive	11091.49	10-yr	237.4	22.32
	11091.49	100-yr	812.6	24.58
Morningside Rd.	483.63	10-yr	236	22.19
	483.63	100-yr	797.1	24.17
Outfall	55.00	10-yr	58.1	22.14
	55.00	100-yr	769.6	23.91

Table 4-7. RDLG Existing Development Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Center	60774.16	10-yr	278.3	31.97
	60774.16	100-yr	420	32.25
Mesquite Grove Subdivision	56205.55	10-yr	556.5	30.74
	56205.55	100-yr	840	31.16
Hwy 77/83	48522.62	10-yr	1799.9	29.82
	48522.62	100-yr	2704.5	30.24
Old Port Isabel	43212.86	10-yr	1507.8	26.59
	43212.86	100-yr	2381.3	27.06
Boca Chica	40705.06	10-yr	1704.4	26.57
	40705.06	100-yr	2323.4	27.04
Btw Boca Chica and 14th	39911.47	10-yr	2666	26.57
	39911.47	100-yr	3742.4	27.04
Willow	31090.42	10-yr	2666	24.12
	31090.42	100-yr	3742.4	26.32
Esperanza	25254.22	10-yr	2209	22.83
	25254.22	100-yr	3508.3	24.92
La Posada	24392.11	10-yr	2209	22.66
	24392.11	100-yr	3508.3	24.86
Southmost	21375.80	10-yr	2843.3	21.08
	21375.80	100-yr	4189.2	22.11
Minnesota Ave.	15463.43	10-yr	2698.3	20.71
	15463.43	100-yr	3850.4	22.21
South Dakota	13087.34	10-yr	2613.7	20.91
	13087.34	100-yr	3772.8	22.5
East of Airport	6525.73	10-yr	1945	20.9
	6525.73	100-yr	2900.9	22.5
Utah	2744.40	10-yr	1935.3	20.69
	2744.40	100-yr	2891.8	22.29

Table 4-8. NMD Existing Development Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Los Ebanos Blvd.	39377.80	10-yr	108	31.5
	39377.80	100-yr	161	31.87
Central Blvd.	37516.04	10-yr	180	31.49
	37516.04	100-yr	270	31.84
Boca Chica Blvd.	35034.97	10-yr	122	30.94
	35034.97	100-yr	243	31.76
Belthair St.	33604.69	10-yr	387	30.94
	33604.69	100-yr	582	31.75
Calle Retama	30282.79	10-yr	387	30.92
	30282.79	100-yr	582	31.73
Palm Blvd.	24605.86	10-yr	72	30.91
	24605.86	100-yr	207	31.72
Old Alice	22576.37	10-yr	222	30.9
	22576.37	100-yr	333	31.71
Ringgold St.	20190.48	10-yr	1217	29.98
	20190.48	100-yr	1837	31.01
US 77/83	16156.75	10-yr	878	29.92
	16156.75	100-yr	1473	30.92
12th St.	15339.09	10-yr	1051	29.85
	15339.09	100-yr	1778	30.78
International Blvd.	10907.08	10-yr	615	27.91
	10907.08	100-yr	615	28.62
East Ave.	4358.43	10-yr	1130	27.04
	4358.43	100-yr	1456	27.66
Calle Milpa Verde Dr.	1620.93	10-yr	1184	25.47
	1620.93	100-yr	1600	26.55
Outlet	242.56	10-yr	1201	24.2
	242.56	100-yr	1629	26.31

Table 4-9_. TR Existing Development Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

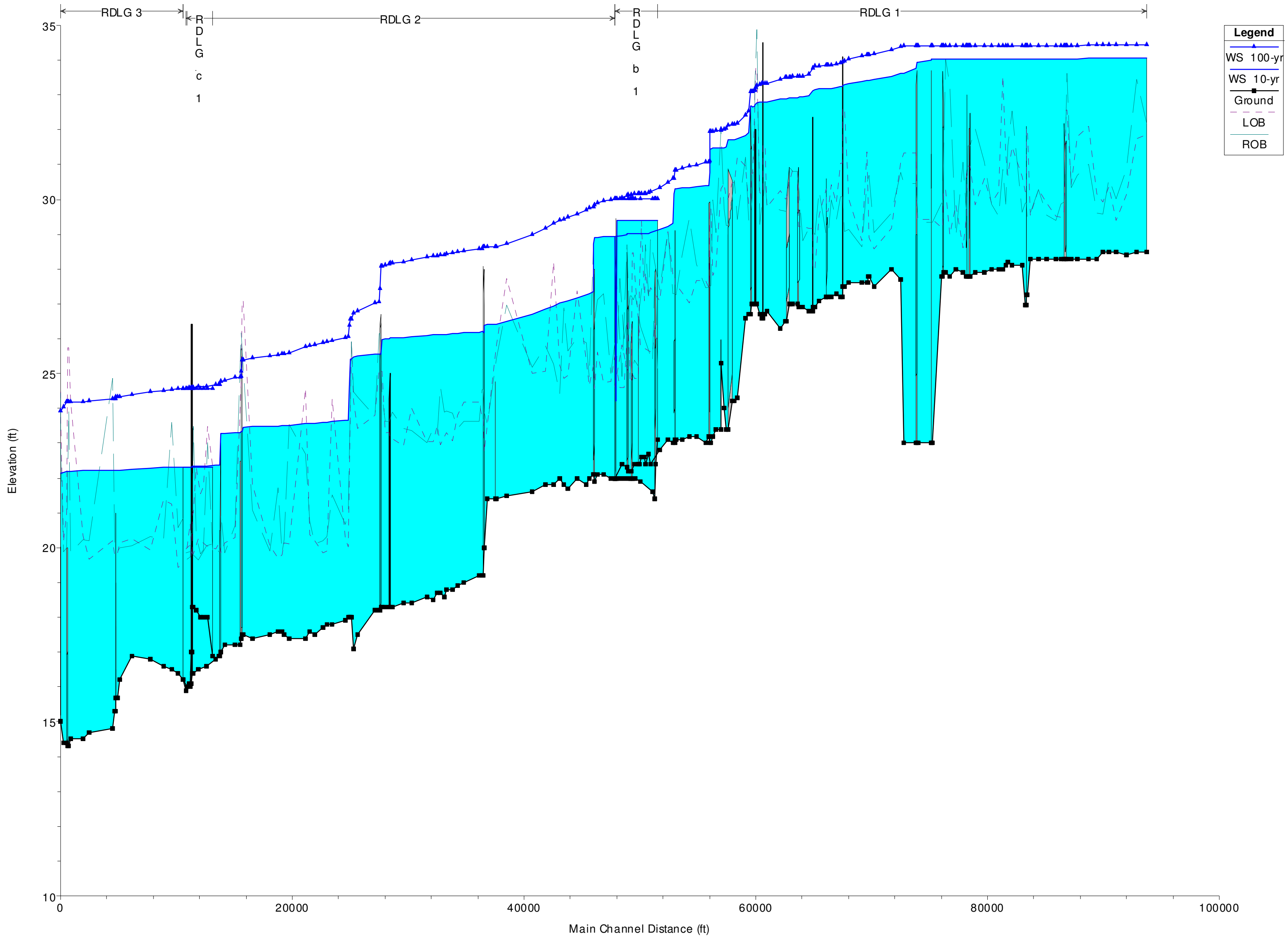

Figure 4-6. Water surface elevation profile for RDLG

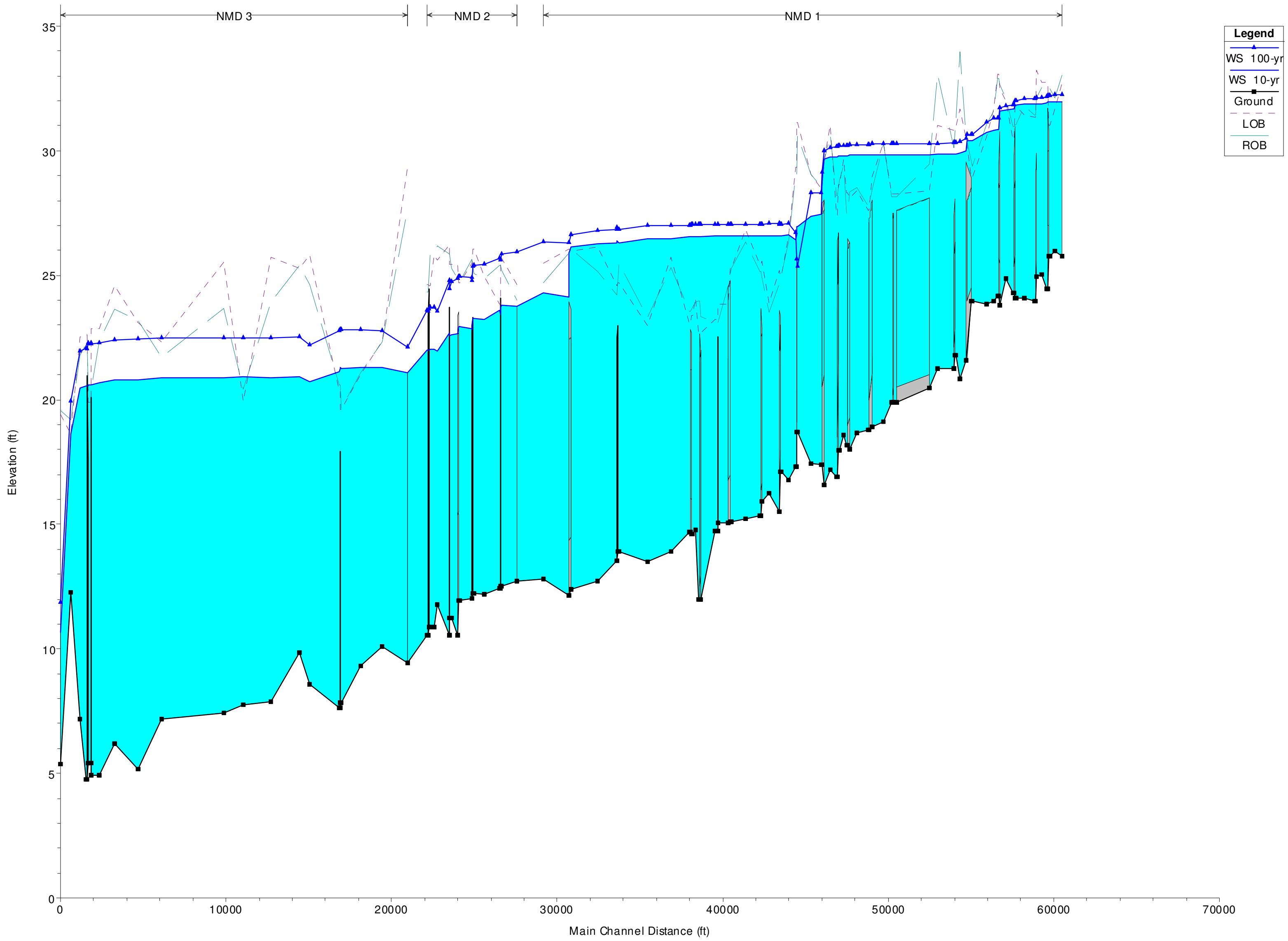

Figure 4-7. Water surface elevation profile for NMD

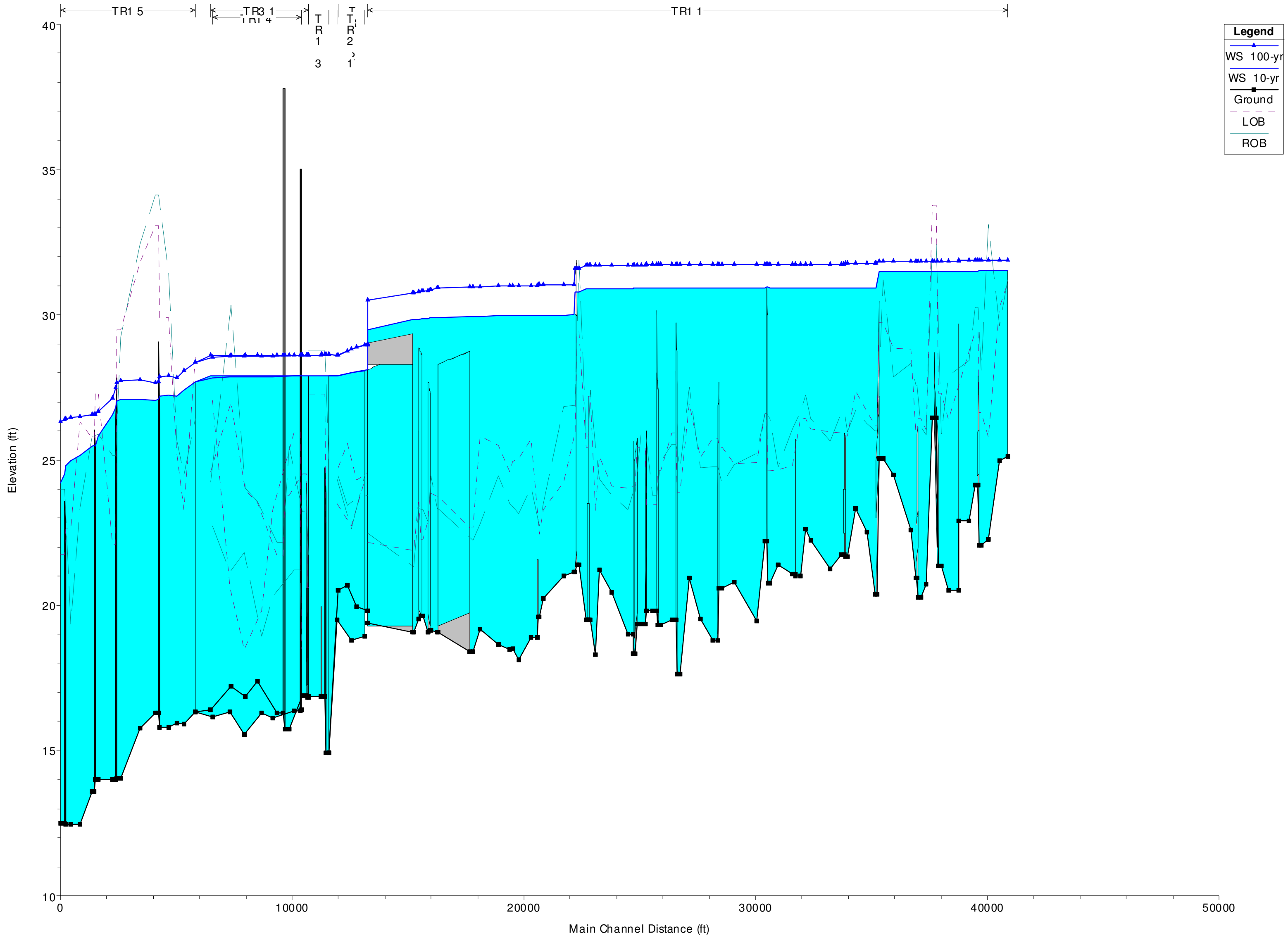

Figure 4-8. Water surface elevation profile for TR

5.0 Storm Surge Analysis

Storm surge is an abnormal rise of water generated by a storm above the predicted astronomical tides. Storm surge is caused by the force of the winds moving cyclonically around the storm pushing the water towards the shore. The maximum potential height of a storm surge depends on a number of factors including storm intensity, forward speed, size, angle of approach to the coast, central pressure, and the shape and characterstics of coastal features such as bays and estuaries and the overall width and slope of the continental shelf (NOAA, 2011). In coastal regions like Brownsville, storm surge often provides the greatest threat to life and property loss from a hurricane. A study completed in 2008 by the U.S. Climate Change Science Program Synthesis and Assessment Product 4.7 (CCSP SAP 4-7) found that a storm surge of 23-ft has the ability to inundate up to 67% of interstates, 57% of arterials, almost half of rail miles, 29 airports, and virtually all ports within the Gulf Coast Area.

One of the largest storm surges that have been experienced in the U.S. in recent history was that observed during Hurricane Katrina between August 23-30, 2005. At that time it was the costliest and one of the five deadliest hurricanes to ever strike the U.S. When Katrina made landfall in the northern Gulf Coast it was a Category 3 storm causing widespread damage and loss of life in Louisiana, Mississippi, Florida, Georgia, and Alabama. While measurement of the storm surge produced by Katrina was very difficult due to widespread failures of tide gauges and the destruction of many buildings along the coast, high-water marks led to the observation of maximum storm surges of 24-28-ft along the Mississippi coast.

Since and prior to Katrina striking the Gulf Coast in 2005 several other hurricanes and tropical waves, depressions, and storms have impacted the Gulf Coast including Dolly in 2008 (Roth, 2010). While Dolly caused several incidences of flooding and damage throughout Cameron County, Brownsville has largely been spared in recent history from any devastating storm activity. In addition, there has been much progress in recent years towards minimizing the impacts of large rains through the cooperation of various entities throughout the county working together to pump down permanent water levels in resacas and drainage ditches prior to large rainfall events. This effort creates additional stormwater capacity within county and city drainage features thus minimizing the impacts of the rainfall. Despite these efforts, Brownsville is and will remain in a coastal zone and as such will be susceptible to impact from hurricanes including large rainfall events and storm surge.

This section of the report attempts to estimate the impacts that a storm surge of Hurricane Katrina magnitude would have on the ability of the City's drainage features to reach each respective outfall. In other words, this section is not identifying every portion of the City that would be subjected to flood waters from an extreme storm surge. Instead, it is looking at the backwater effect that such a surge would have on local drainage ditches and resacas to drain overland runoff from the City during extreme storm surge conditions and a given rainfall frequency. To complete this analysis a boundary condition was applied to the HEC-RAS model and then the model was run with various flows, representative of rainfall from the 6 frequency events previously discussed. The assumed boundary condition value used to complete this analysis was the maximum storm surge observed during Hurricane Katrina of 28-ft. It should be noted that Hurricane Katrina was an extreme and rare event and is not likely from a probability standpoint to be witnessed again for many years. However, this section is meant to illustrate a "worst case" scenario.

5.1 Storm Surge Impact on Upper RRV and CCDD3

The upper RRV and CCDD3 watersheds are the furthest west from the coast of all those discussed in this report. However, the much of the area is still very low in elevation especially in the area surrounding CCDD3 where the impact of the storm surge is significant. Overall, for a 10-yr rainfall frequency event with the assumed 28-ft storm surge the combined URRV/CCDD3 watershed area was inundated with floodwaters on over 30% of the entire land area (11,328 acres)(Figure 5-1). While differences in overall water surface elevations were negligible in the URRV portion of the watershed, in CCDD3 between nearly 3-ft to 15-ft of increased flood elevations were observed. For the 100-yr rainfall event with the assumed 28-ft storm surge nearly 35% of the entire combined land area was inundated with flood waters (12,928 acres) (Figure 5-2) resulting in 2.5-ft to nearly 16-ft of increased water surface elevations as compared to the existing development conditions without the storm surge component. The results of this analysis for both the 10-yr and 100-yr frequency rainfall events, with storm surge, including the computed water surface elevations may be observed in Table 5-2. The numbers of buildings predicted to be impacted for both the 10-yr and 100-yr event are 1432 and 1620 respectively and are presented in Table 5-1.

	Surge Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	139	162
0.5-1	92	140
1-1.5	134	152
1.5-2	151	167
2-2.5	153	165
2.5-3	133	140
3-3.5	133	145
3.5-4	146	162
4-4.5	114	129
4.5-5	85	96
5.0-8	113	123
8.0-12	37	37
>12.0	2	2
Total	1432	1620

Table 5-1. Number of Structures/Buildings predicted to be flooded in the URRV/CCDD3 watershed model under maximum Hurricane Katrina storm surge conditions for the 10-yr and 100-yr frequency rainfall

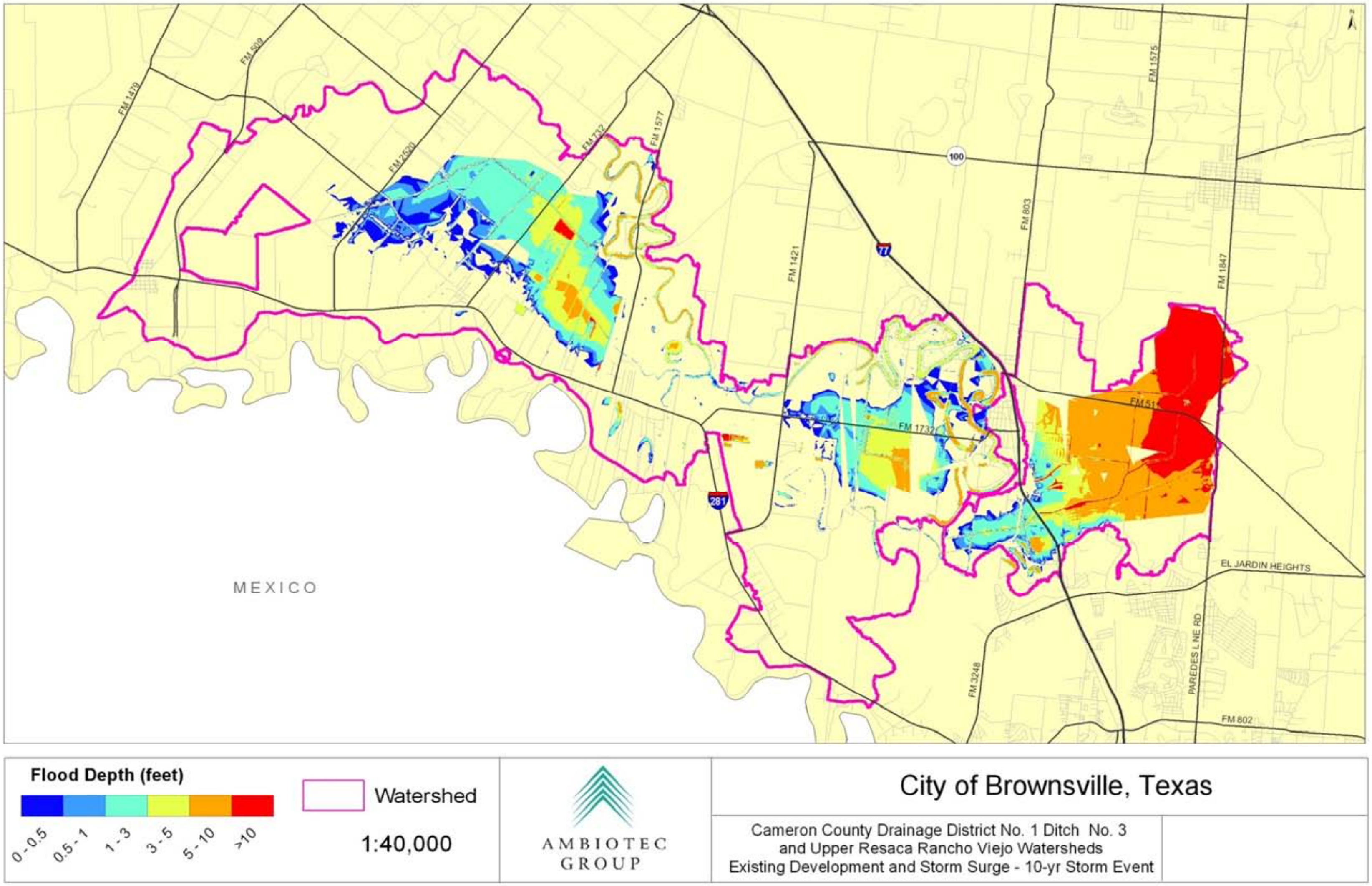

Figure 5-1. Floodplain for the URRV/CCDD3 watersheds under extreme storm surge conditions (28-ft) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

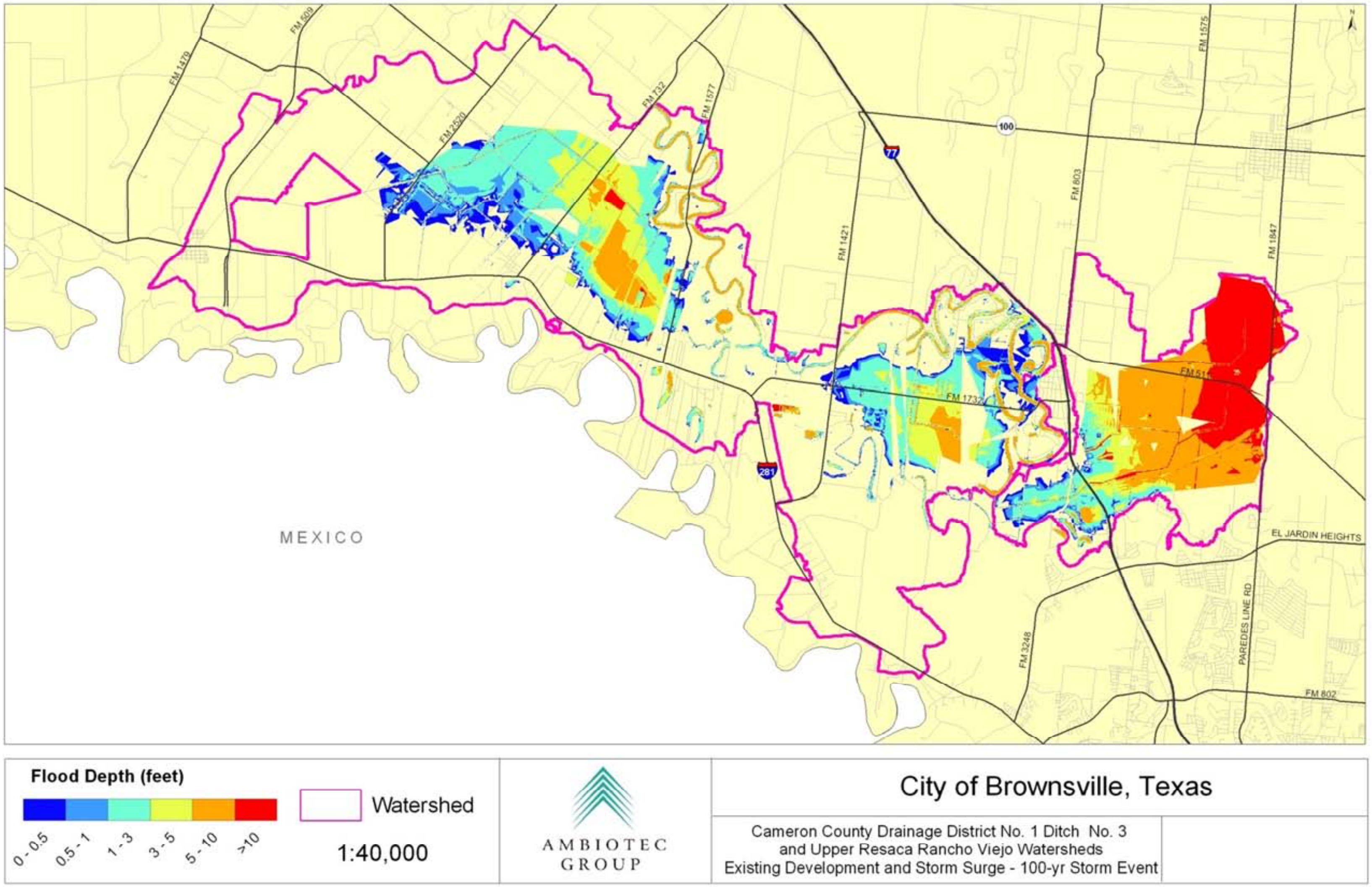

Figure 5-2. Floodplain for the URRV/CCDD3 watersheds under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
FM 732 (URRV)	236667.20	10-yr	19.3	43.44
	236667.20	100-yr	33.3	44.43
FM 1577 (1st crossing)(URRV)	225636.00	10-yr	69.4	43.44
	225636.00	100-yr	109.6	44.42
Rice Tract Rd. (URRV)	202893.80	10-yr	1321.7	39.3
	202893.80	100-yr	2894.6	41.75
Near Guagolota Rd. (URRV)	188108.40	10-yr	1062	36.16
	188108.40	100-yr	2384.7	36.91
FM 1421 / Barreda Garden Rd. (URRV)	179139.60	10-yr	1075.4	35.11
	179139.60	100-yr	2452.2	36.57
Escandon Ave. (URRV)	160701.80	10-yr	798.1	34.41
	160701.80	100-yr	1804.2	35.19
Balboa Rd. (URRV)	149664.30	10-yr	816	34.3
	149664.30	100-yr	1881.4	34.73
FM 1732 / Cavazons Olmito Rd. (URRV)	128689.10	10-yr	772.6	31.21
	128689.10	100-yr	1875	32.61
Near Lakeside Blvd. 1 (URRV)	120539.90	10-yr	772.6	31.17
	120539.90	100-yr	1875	32.47
Near Lakeside Blvd. 2 (from Resaca to Ditch)	111686.00	10-yr	2308.4	30.52
	111686.00	100-yr	3917.9	31.16
Before Overflow Structure to CCDD3	110760.00	10-yr	2308.4	29.58
	110760.00	100-yr	3917.9	30.12
Railroad (CCDD3)	27014.17	10-yr	2491.8	28.04
	27014.17	100-yr	4306.8	28.09
US 77/83 (CCDD3)	25093.72	10-yr	2573.4	28.01
	25093.72	100-yr	4476.9	28.03
Old Alice Rd (CCDD3)	22959.53	10-yr	2573.4	28
	22959.53	100-yr	4476.9	28.01
Box Culvert after Ditch Junction (CCDD3)	15679.25	10-yr	1057.4	28
	15679.25	100-yr	1638.7	28
FM 511 (CCDD3)	8888.71	10-yr	1746	28
	8888.71	100-yr	2798.5	28
Undeveloped Area Before Outfall	5438.58	10-yr	2169.1	28
	5438.58	100-yr	3579.5	28
Outfall	317.46	10-yr	2167.9	28
	317.46	100-yr	3638.8	28

Table 5-2. URRV/CCDD3 Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

5.2 Storm Surge Impact on Lower RRV

As compared to the URRV/CCDD3 watershed area, the outfall for the LRRV watershed is closer to the coast thus the elevation is lower and the impact of the storm surge more profound. An assumed storm surge of 28-ft was again set and combined with different frequency rainfall events. Through this analysis it was observed that a 10-yr rainfall frequency with an assumed 28-ft storm surge resulted in the flooding of approximately 44% of the entire LRRV watershed land area (1,777 acres) (Figure 5-3). For the 100-yr rainfall event nearly 47% (1,875 acres) of the entire land area was flooded (Figure 5-4). As compared to the flooding experienced for each rainfall frequency event without the storm surge component, this is a significant increase in floodplain area and results in a significant increase in the number of buildings expected to flood (1048 and 1165 for the 10-yr and 100-yr events respectively). Table 5-3 shows the computed number of buildings expected to flood within given water depth ranges for the 10-yr and 100-yr rainfall events under the assumed storm surge conditions. The computed water surface elevations (W.S.E.) and assumed flows may be observed in Table 5-4. When comparing the computed water surface elevations to that of the existing conditions without the assumed storm surge, it may be observed that the surge resulted approximately in an additional 2-ft to 23-ft of increased water depths for the 10-yr rainfall frequency and between approximately 0.14-ft to 23-ft for the 100-yr. Overall, it may be observed that the impacts of the storm surge were more sever on the smaller (10-yr) frequency event than that of the 100-yr. Essentially, as rainfall intensities get higher and cause more flooding, the impacts of storm surge are masked as compared to that of smaller rainfall frequency events.

	Surge Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	126	177
0.5-1	139	169
1-1.5	130	150
1.5-2	88	96
2-2.5	97	104
2.5-3	110	108
3-3.5	108	110
3.5-4	92	93
4-4.5	57	58
4.5-5	53	53
5.0-8	46	45
8.0-12	2	2
>12.0	0	0
Total	1048	1165

Table 5-3. Number of Structures/Buildings predicted to be flooded in the LRRV watershed model under maximum Hurricane Katrina storm surge conditions for the 10-yr and 100-yr frequency rainfall

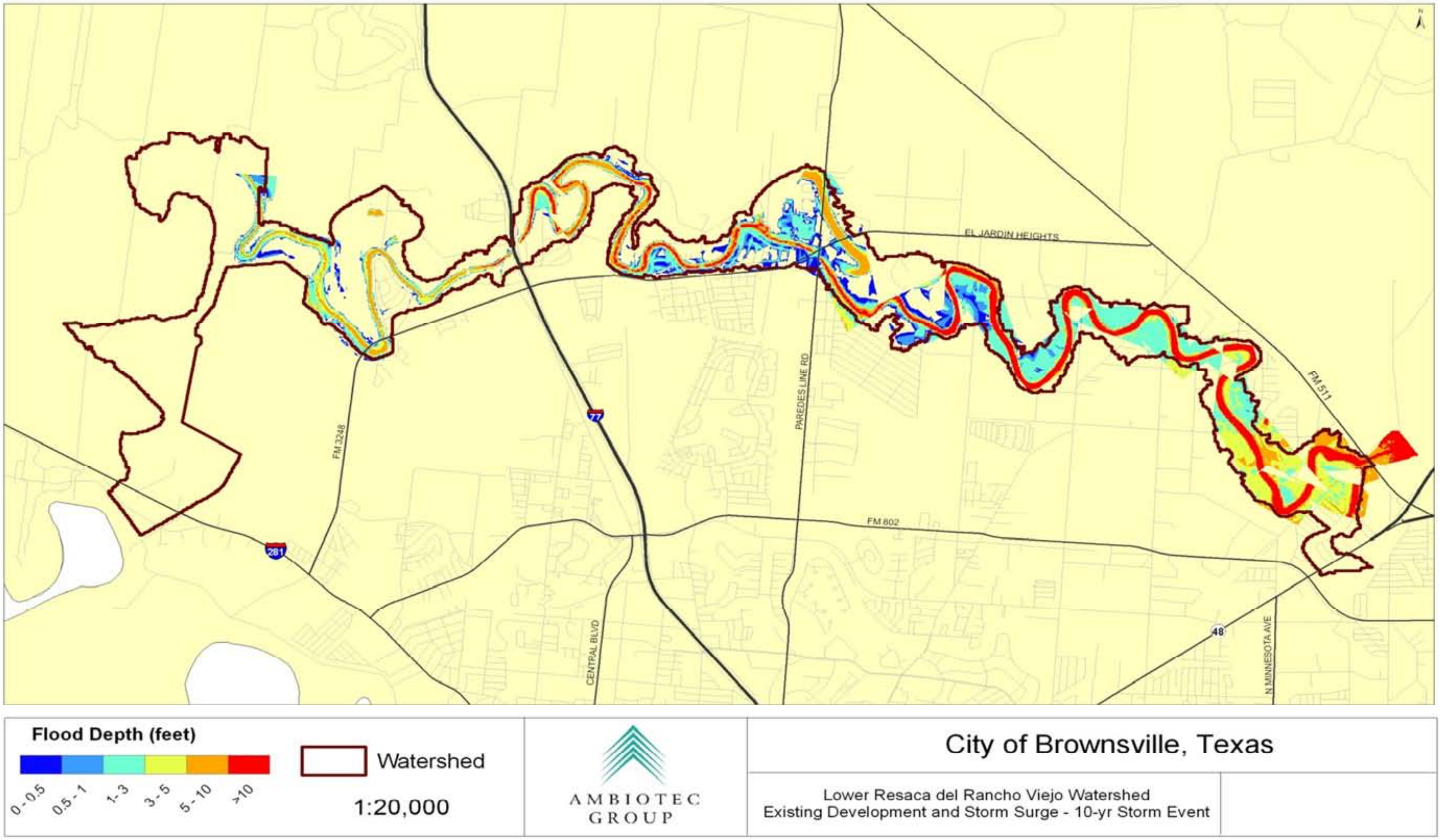

Figure 5-3. Floodplain for the LRRV watersheds under extreme storm surge conditions (28-ft) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

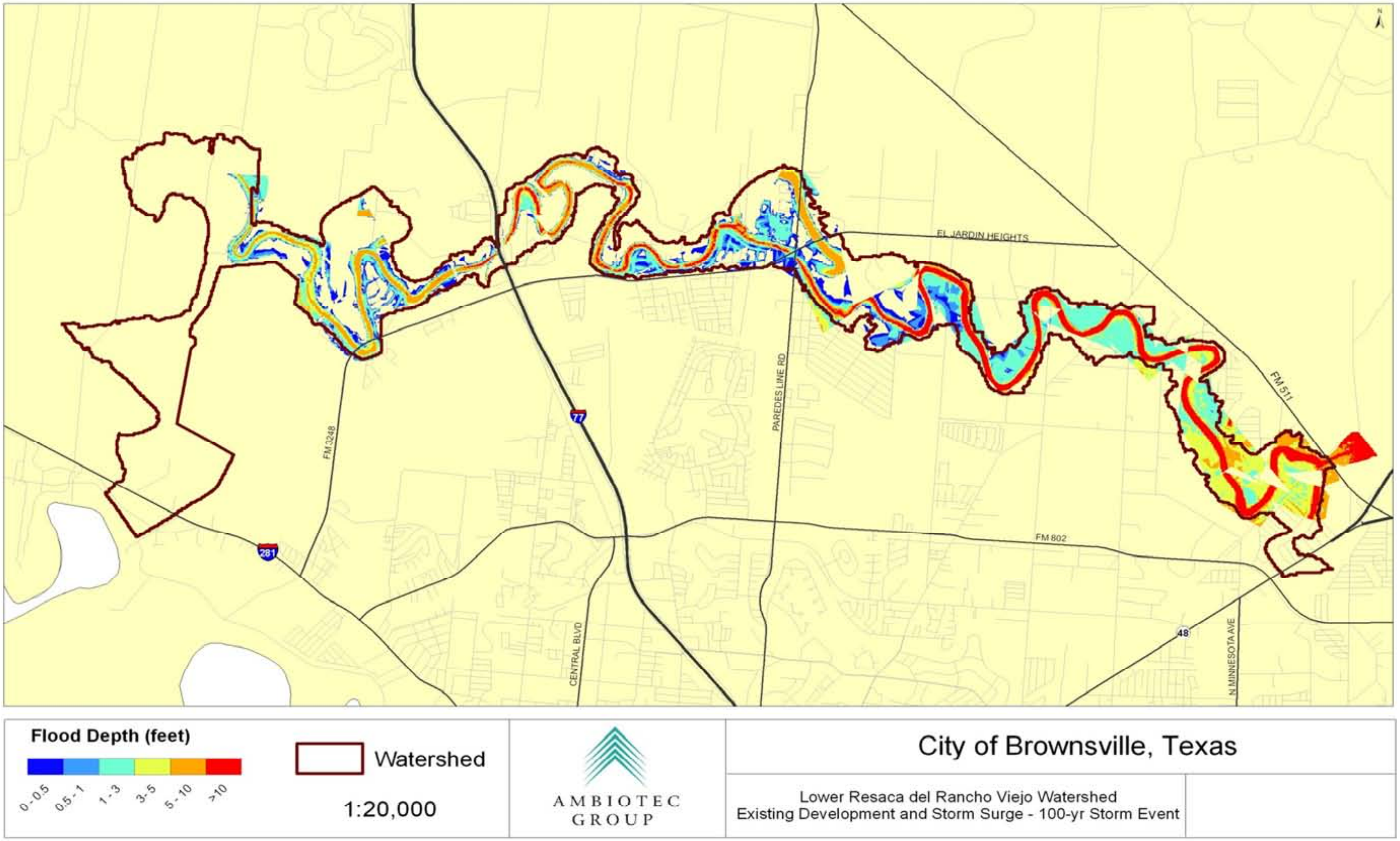

Figure 5-4. Floodplain for the LRRV watersheds under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Furthest Upstream Area	99344.57	10-yr	143.9	31.85
	99344.57	100-yr	230.7	32.3
Alton Gloor (FM 3248)	86422.92	10-yr	204.7	31.8
	86422.92	100-yr	364	32.16
Lakeway Drive	81775.41	10-yr	196.9	28.79
	81775.41	100-yr	359.7	30.86
Downstream of US 77/83	74557.76	10-yr	106.6	28.05
	74557.76	100-yr	165.5	28.12
Dennett Rd./Stillman Rd.	62008.83	10-yr	199.9	28.03
	62008.83	100-yr	367	28.08
Stagecoach Trail	53542.97	10-yr	188.3	28.03
	53542.97	100-yr	343	28.06
Paredes Line Rd.	47681.73	10-yr	181.5	28.01
	47681.73	100-yr	351	28.01
Dana Rd.	36180.49	10-yr	75.3	28.01
	36180.49	100-yr	312.7	28.01
Robindale Rd.	25334.66	10-yr	113.6	28.01
	25334.66	100-yr	250.3	28
Old Port Isabel Rd.	21421.27	10-yr	115.6	28
	21421.27	100-yr	185.1	28
NW of Morrison Rd. and Salida del Sol	13058.5	10-yr	115.6	28
	13058.5	100-yr	185.1	28
Charmaine Rd.	6957.445	10-yr	161	28
	6957.445	100-yr	258.9	28
Near Railroad Crossing before Ditch	3144.294	10-yr	161	28
	3144.294	100-yr	258.9	28
Outfall (Ditch)	298.6842	10-yr	87.9	28
	298.6842	100-yr	138.7	28

Table 5-4. LRRV Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

5.3 Storm Surge Impact on CCDD1

The effect of a 28-ft storm surge was again analyzed for the CCDD1 watershed. The CCDD1 watershed was part of the 2006 study and not modified during this phase II study. The 2006 study revealed large flood-prone areas throughout the watershed and recommended several mitigation strategies to alleviate flooding (which will be updated in Section 8). The results of the storm surge analysis on this watershed revealed that for both the 10-yr and 100-yr rainfall events combined with the assumed 28-ft surge, approximately 70% of the entire watershed area (10,286 acres and 10,297 acres respectively) would be inundated with water. However, in examining the floodplains for both frequency events (Figures 5-5 & 5-6) along with elevation data throughout the entire watershed area it could be assumed that a larger area would be impacted by the storm surge. Additionally, it may be observed that there is very little difference between floodplains from the 10-yr to the 100-yr frequency event and is further supported by examining the water surface elevations for each storm in Table 5-6. The difference in computed water surface elevations between the 10-yr and 100-yr rainfall events is below 0.10-ft across the watershed illustrating the impact that an extreme storm surge would have on local floodplains, masking the impact of rainfall volume alone. This scenario is predicted to impact nearly 16,000 buildings for both the 10-yr and 100-yr rainfall frequencies under extreme storm surge conditions. The summary of expected buildings impacted by varying flood depth levels is presented in Table 5-5.

	Surge Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	314	295
0.5-1	354	362
1-1.5	347	353
1.5-2	408	404
2-2.5	524	500
2.5-3	405	431
3-3.5	386	386
3.5-4	475	458
4-4.5	677	657
4.5-5	852	842
5.0-8	6298	6305
8.0-12	4382	4430
>12.0	274	296
Total	15696	15719

Table 5-5. Number of Structures/Buildings predicted to be flooded in the CCDD1 watershed model under maximum Hurricane Katrina storm surge conditions for the 10-yr and 100-yr frequency rainfall

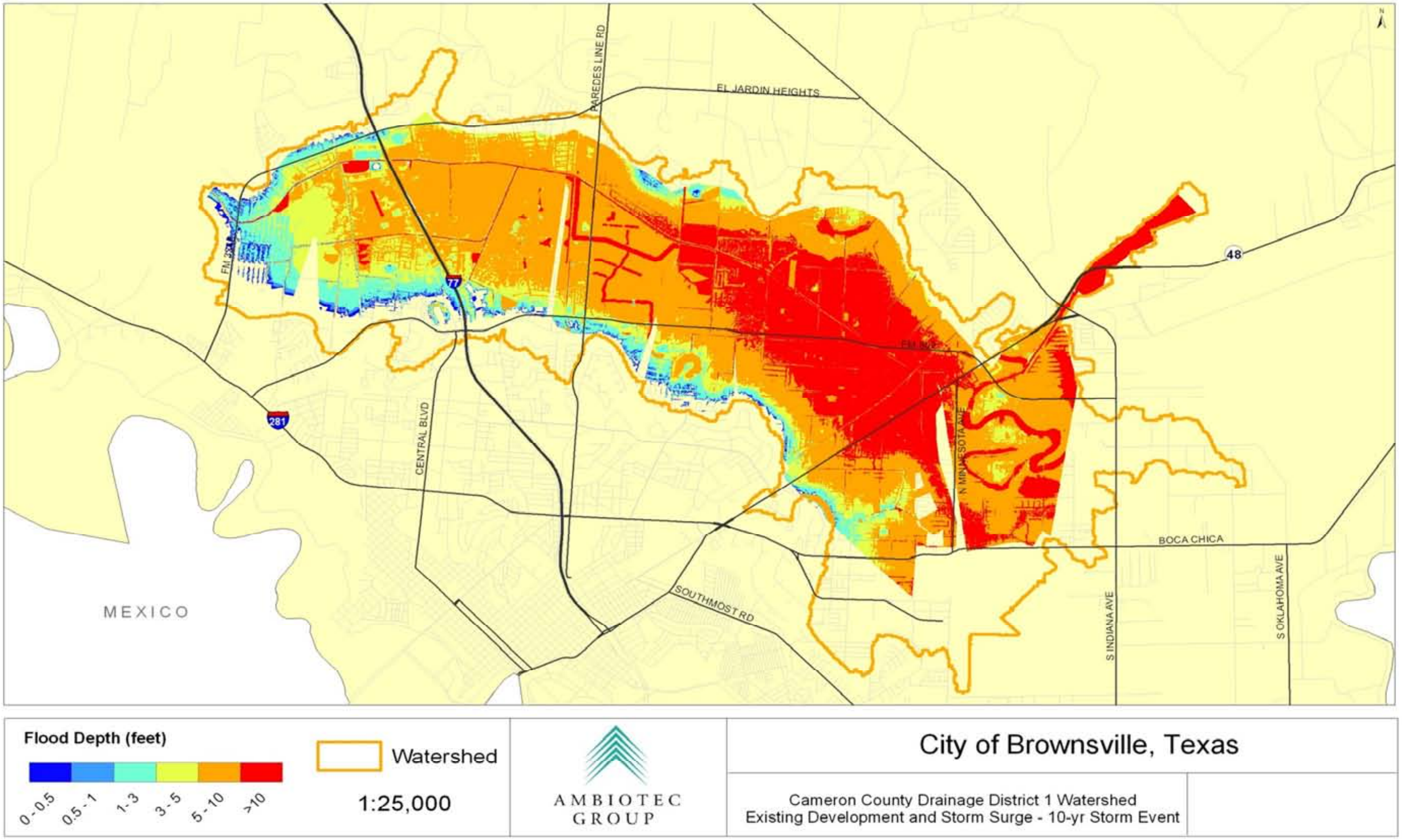

Figure 5-5. Floodplain for the CCDD1 watershed under extreme storm surge conditions (28-ft) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

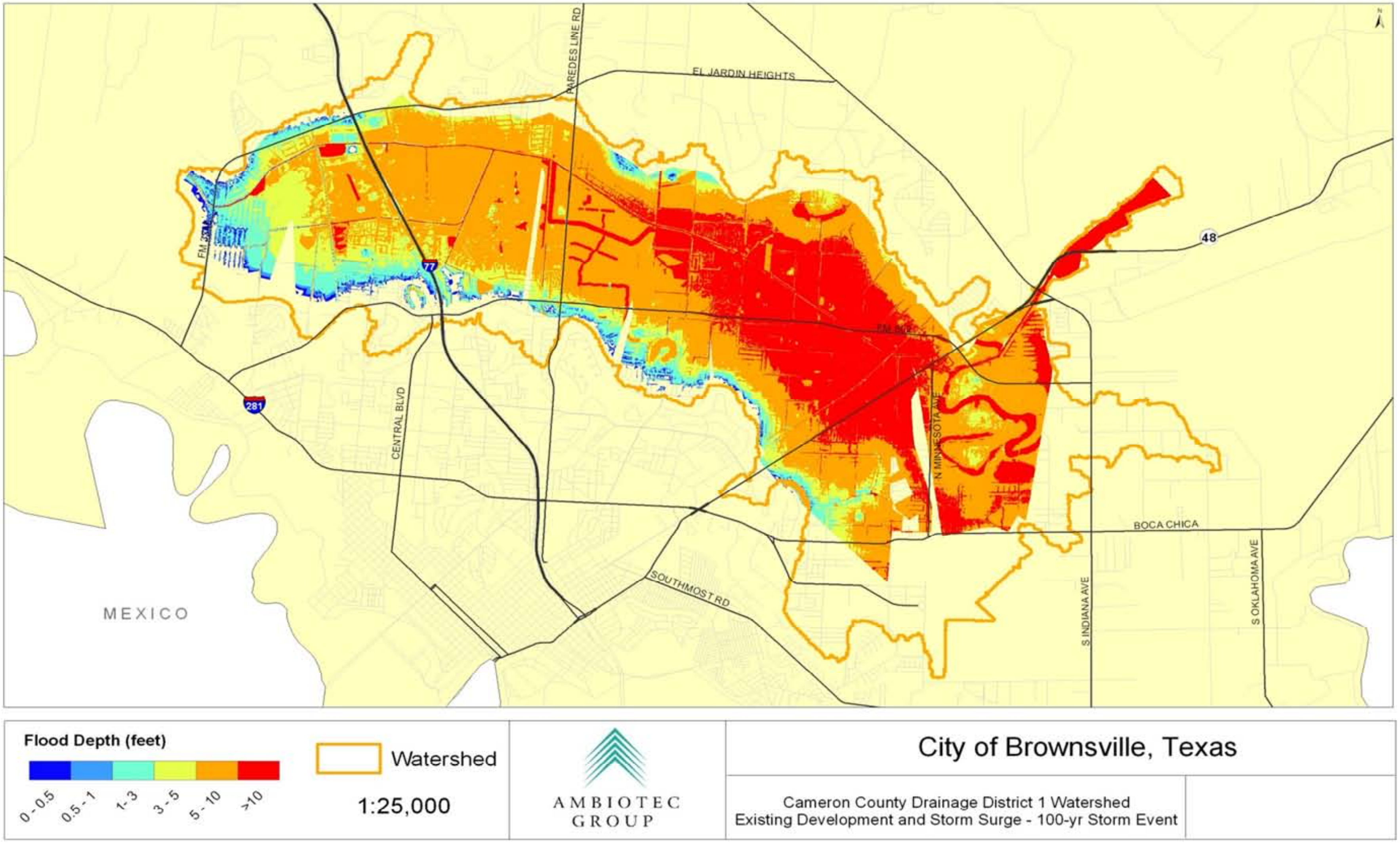

Figure 5-6. Floodplain for the CCDD1 watershed under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (exist.- 2006 study)	W.S.E. (ft)
Beginning	59386.47	10-yr	1	27.24	28.06
	59386.47	100-yr	1	27.35	28.12
Alton Gloor	57023.95	10-yr	224	22.94	28.06
	57023.95	100-yr	354	23.45	28.11
US 77/83	48213.56	10-yr	672	21.39	28.06
	48213.56	100-yr	1061.9	21.86	28.11
Pablo Kisel	44494.17	10-yr	896	20.94	28.06
	44494.17	100-yr	1415.8	21.42	28.11
Paredes Line Rd.	37764.99	10-yr	500	17.77	28.06
	37764.99	100-yr	800	18.72	28.11
Dana Ave.	30766.94	10-yr	2810.2	17.32	28.06
	30766.94	100-yr	4353.3	18.41	28.1
Old Port Isabel	28290.97	10-yr	2810.2	16.98	28.06
	28290.97	100-yr	4353.3	18.2	28.1
Robindale	26203.97	10-yr	2810.2	16.9	28.06
	26203.97	100-yr	4353.3	18.2	28.1
Central Ave	22326.12	10-yr	3051.7	16.8	28.06
	22326.12	100-yr	4550.6	18.13	28.1
FM 802	21389.29	10-yr	3051.7	16.78	28.06
	21389.29	100-yr	4550.6	18.11	28.1
Hwy 48 & Minnesota	16499.11	10-yr	4464.3	16.35	28.05
	16499.11	100-yr	6044.9	17.99	28.1
FM 802	14785.25	10-yr	4612.3	16	28.05
	14785.25	100-yr	6099.6	17.77	28.1
Capt. Donald L. Faust	10256.28	10-yr	4779.8	11.36	28.02
	10256.28	100-yr	6342.6	12.89	28.03
FM 511	9272.44	10-yr	4779.8	10.94	28.02
	9272.44	100-yr	6342.6	12.51	28.03
Hwy 48	6391.51	10-yr	4779.8	9.12	28
	6391.51	100-yr	6342.6	10.22	28

Table 5-6. CCDD1 Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

5.4 Storm Surge Impact on the Merged RDLG, NMD, & TR Watersheds

The results of the storm surge analysis that was completed on the merged RDLG, NMD, and TR HEC-RAS model revealed flooding on over 42% of the entire combined land area (for all three watersheds) for the 10-yr event and about 45% for the 100-yr event (5408 acres and 5760 acres respectively). The resulting floodplains caused by the 28-ft storm surge with both the 10-yr and 100-yr rainfall frequencies are presented in Figures 5-7 & 5-8. The impact of the storm surge resulted in an increase of flood depths of between 0 and nearly 7.5-ft throughout the combined watershed area. The computed water surface elevations for each watershed area analyzed in the merged model are presented in Tables 5-8 through 5-10. The largest increases in projected water surface elevations for the storm surge scenario relative to the existing conditions scenario were observed along NMD with higher impact seen further downstream. Water surface elevations along RDLG were elevated from existing conditions by up to 5-ft but not as significantly as along NMD. For both watersheds the main impact was observed downstream of Old Port Isabel Rd. Upstream of this point, little deviation was observed in computed water surface elevations from the storm surge as compared with the existing development without surge analysis. Along TR modest increase in computed water surface elevations were observed as far upstream as Calle Retama (less than 0.5-ft) however deviations of 1-ft or greater were not observed until much further downstream. In the vicinity of East Avenue down to the outfall into NMD, computed water surface elevations were approximately 2-4-ft higher as those computed during existing conditions without storm surge. Overall, a storm of this magnitude is predicted to impact approximately 7240 buildings for the 10-yr frequency event and 7890 for the 100-yr. The number of expected flooded buildings within varying flood depth ranges is presented in Table 5-7.

	Surge Conditions	
Water Depth (ft)	**10-Yr**	**100-Yr**
0-0.5	1039	1252
0.5-1	1036	1115
1-1.5	967	1030
1.5-2	985	1007
2-2.5	955	997
2.5-3	758	795
3-3.5	476	575
3.5-4	263	299
4-4.5	171	197
4.5-5	157	155
5.0-8	423	457
8.0-12	10	11
>12.0	0	0
Total	7240	7890

Table 5-7. Number of Structures/Buildings predicted to be flooded in the merged watershed model (RDLG, NMD, and TR) under maximum Hurricane Katrina storm surge conditions for the 10-yr and 100-yr frequency rainfall

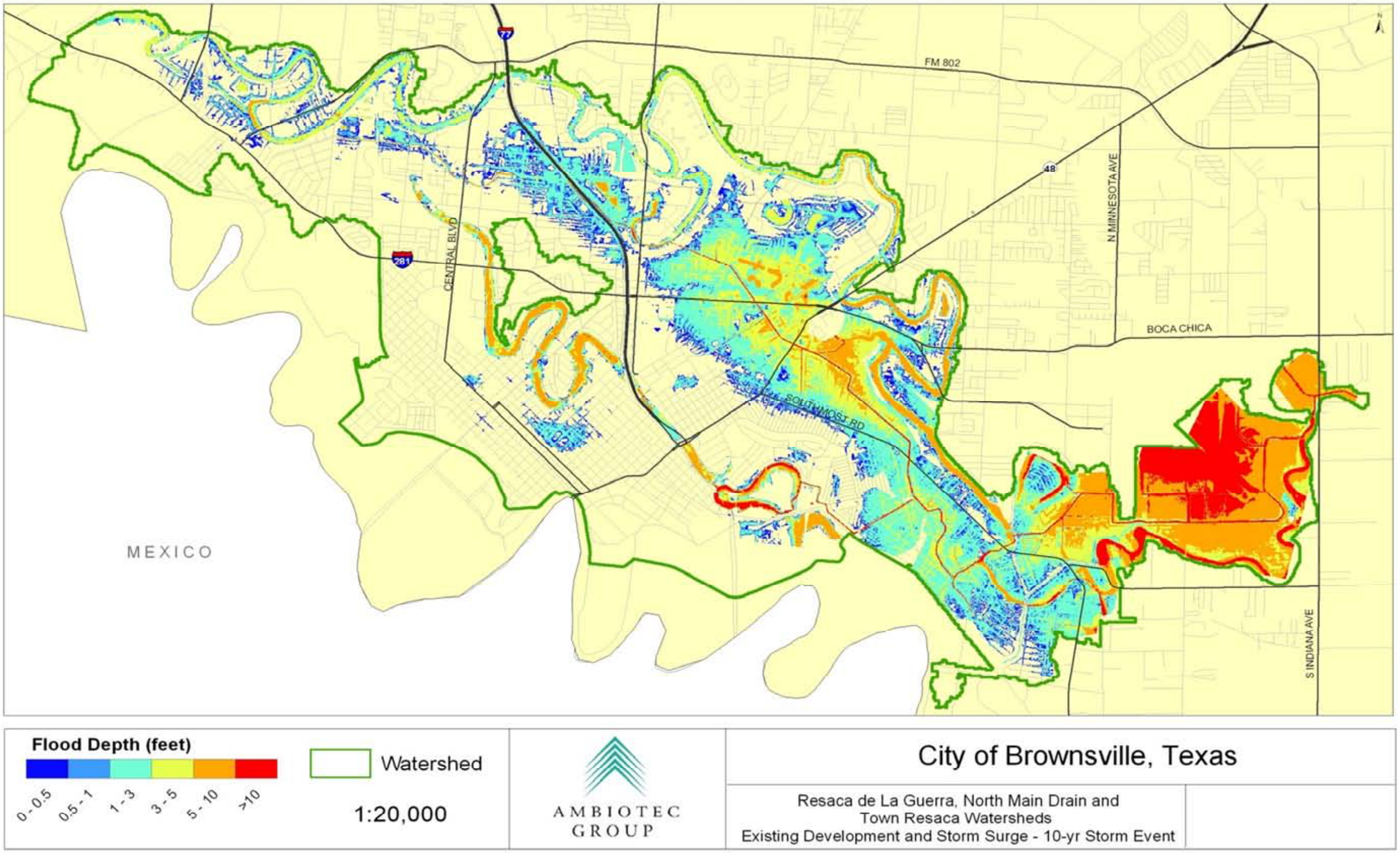

Figure 5-7. Floodplain for the merged model (RDLG, NMD, and TR watersheds) under extreme storm surge conditions (28-ft) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

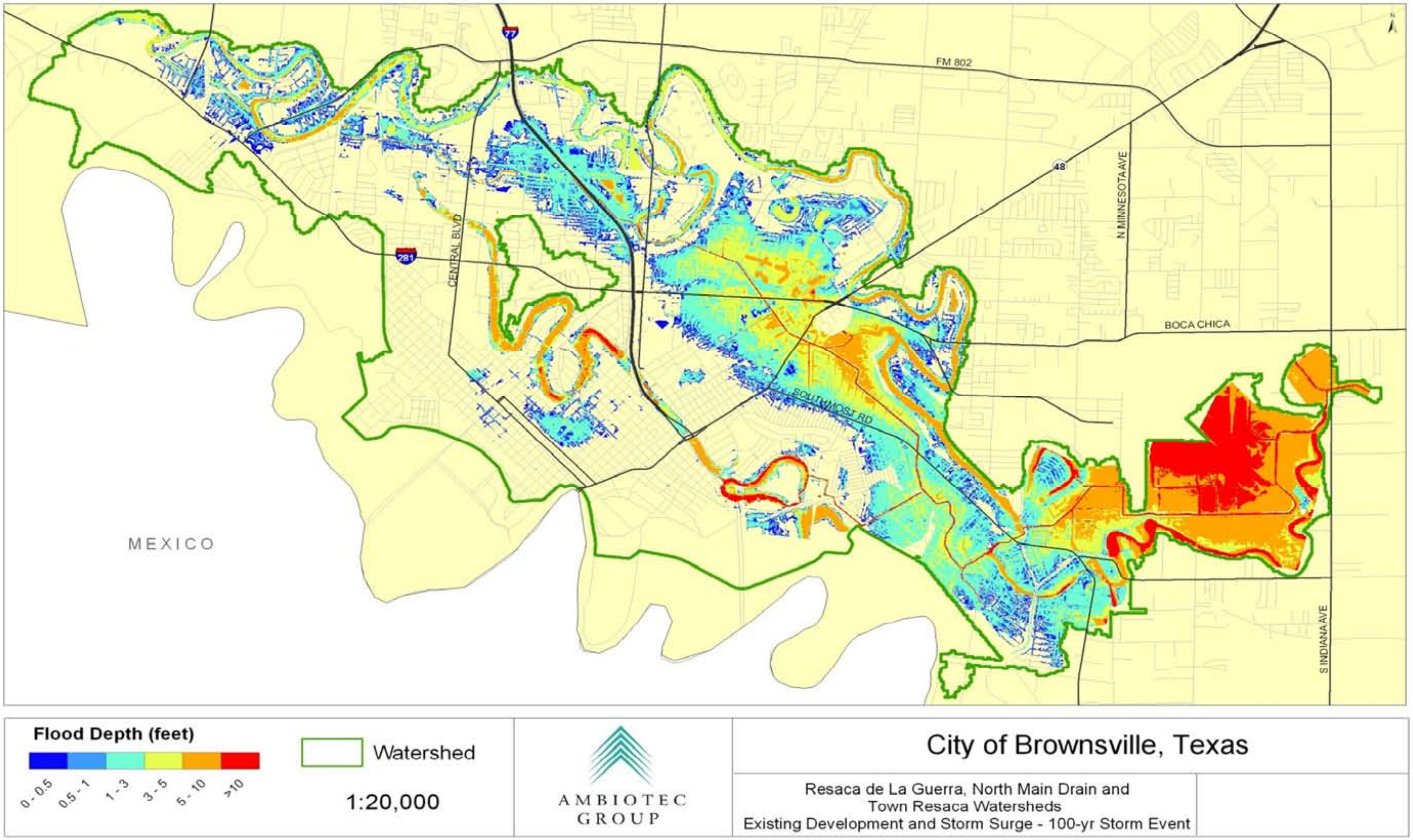

Figure 5-8. Floodplain for the merged model (RDLG, NMD, and TR watersheds) under extreme storm surge conditions (28-ft) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Alton Gloor	84480.30	10-yr	3.8	34.04
	84480.30	100-yr	18.3	34.44
Laredo Rd.	75741.20	10-yr	105.1	34.04
	75741.20	100-yr	162.5	34.43
Golf Course (upstream VICC)	67843.07	10-yr	654.9	33.24
	67843.07	100-yr	999.9	33.95
Central Blvd.	59913.97	10-yr	575	31.91
	59913.97	100-yr	1012.7	32.54
US 77/83	58006.56	10-yr	575	31.47
	58006.56	100-yr	1012.7	32.03
Old Alice	56431.98	10-yr	528.9	30.41
	56431.98	100-yr	1041.5	31.14
Paredes Line Rd.	49414.23	10-yr	421.1	29.04
	49414.23	100-yr	1024.5	30.3
Old Port Isabel	36395.52	10-yr	287.1	28.16
	36395.52	100-yr	956.3	28.99
Price Rd.	27465.97	10-yr	287.1	28.1
	27465.97	100-yr	956.3	28.57
Hwy 48	24796.52	10-yr	242.5	28.08
	24796.52	100-yr	856.5	28.44
Boca Chica	15543.40	10-yr	242.5	28.06
	15543.40	100-yr	856.5	28.28
Billy Mitchell Blvd.	13591.11	10-yr	237.4	28.05
	13591.11	100-yr	812.6	28.23
Acacia Lake Drive	11091.49	10-yr	237.4	28.05
	11091.49	100-yr	812.6	28.21
Morningside Rd.	483.63	10-yr	236	28.03
	483.63	100-yr	797.1	28.13
Outfall	55.00	10-yr	58.1	28.02
	55.00	100-yr	769.6	28.05

Table 5-8. RDLG Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Center	60774.16	10-yr	278.3	31.96
	60774.16	100-yr	420	32.25
Mesquite Grove Subdivision	56205.55	10-yr	556.5	30.71
	56205.55	100-yr	840	31.16
Hwy 77/83	48522.62	10-yr	1799.9	29.89
	48522.62	100-yr	2704.5	30.17
Old Port Isabel	43212.86	10-yr	1507.8	28.16
	43212.86	100-yr	2381.3	28.35
Boca Chica	40705.06	10-yr	1704.4	28.15
	40705.06	100-yr	2323.4	28.34
Btw Boca Chica and 14th	39911.47	10-yr	2666	28.15
	39911.47	100-yr	3742.4	28.34
Willow	31090.42	10-yr	2666	28.09
	31090.42	100-yr	3742.4	28.21
Esperanza	25254.22	10-yr	2209	28.04
	25254.22	100-yr	3508.3	28.09
La Posada	24392.11	10-yr	2209	28.03
	24392.11	100-yr	3508.3	28.07
Southmost	21375.80	10-yr	2843.3	28.02
	21375.80	100-yr	4189.2	28.04
Minnesota Ave.	15463.43	10-yr	2698.3	28.02
	15463.43	100-yr	3850.4	28.03
South Dakota	13087.34	10-yr	2613.7	28.02
	13087.34	100-yr	3772.8	28.04
East of Airport	6525.73	10-yr	1945	28.02
	6525.73	100-yr	2900.9	28.04
Utah	2744.40	10-yr	1935.3	28.01
	2744.40	100-yr	2891.8	28.03

Table 5-9. NMD Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)
Los Ebanos Blvd.	39377.80	10-yr	108	31.53
	39377.80	100-yr	161	31.87
Central Blvd.	37516.04	10-yr	180	31.52
	37516.04	100-yr	270	31.82
Boca Chica Blvd.	35034.97	10-yr	122	31.13
	35034.97	100-yr	243	31.77
Belthair St.	33604.69	10-yr	387	31.12
	33604.69	100-yr	582	31.77
Calle Retama	30282.79	10-yr	387	31.1
	30282.79	100-yr	582	31.74
Palm Blvd.	24605.86	10-yr	72	31.1
	24605.86	100-yr	207	31.73
Old Alice	22576.37	10-yr	222	31.09
	22576.37	100-yr	333	31.72
Ringgold St.	20190.48	10-yr	1217	30.29
	20190.48	100-yr	1837	31.05
US 77/83	16156.75	10-yr	878	30.25
	16156.75	100-yr	1473	30.97
12th St.	15339.09	10-yr	1051	30.18
	15339.09	100-yr	1778	30.84
International Blvd.	10907.08	10-yr	615	28.67
	10907.08	100-yr	615	29.04
East Ave.	4358.43	10-yr	1130	28.22
	4358.43	100-yr	1456	28.41
Calle Milpa Verde Dr.	1620.93	10-yr	1184	28.1
	1620.93	100-yr	1600	28.22
Outlet	242.56	10-yr	1201	28.09
	242.56	100-yr	1629	28.2

Table 5-10. TR Storm Surge Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

6.0 Analysis of Additional Candidate Drainage Mitigation Strategies and Updated CIP

The 2006 Flood Study laid out a series of 5 year Capital Improvement Plans (CIPs) to be implemented over a 20-yr period to help minimize the damages from an expected rainfall event. These CIPs included several structural and non-structural flood mitigation strategies including the construction of numerous detention ponds, the widening and concrete lining of major drainage ditches and the implementation of runoff controls. While the main purpose of this study was to incorporate regions of the study that have previously not been modeled to assess baseline flooding conditions in portions of the City that is likely to experience future growth, another aspect of the study was to examine any additional structural improvements that may mitigate local flooding issues, and to examine an inter-watershed transfer of stormwater flow. This section of the report will address two additional mitigation strategies that were investigated. Section 6.1 will discuss the incorporation of several culvert improvements within the merged RDLG, NMD, and TR watershed system and Section 6.2 will discuss a transfer of stormwater flow from NMD to RDLG. The original idea that was discussed during the 2006 study was to transfer stormwater from CCDD1 to the LRRV however, several factors have made that scenario undesirable including cost and right-of-way issues. Additionally, some issues of repeat street flooding events on the downstream segments of LRRV have made this idea undesirable to many local residents and local agencies that manage flows within the Resaca. Several meetings with local drainage regulating entities have revealed concerns about augmenting that Resaca with any additional flow.

6.1 Culvert Improvements along RDLG (Option 1)

Several additional culvert improvements were analyzed along RDLG where old, under-sized culverts have caused issues with backwater effects. The specific locations of the proposed culvert improvements along with existing and improved culvert types and sizes may be viewed in Table 6-1.

Street	Existing Culvert	Improved Culvert
Shidler Rd.	2-ft RCP	4-ft RCP
Price Rd	2-ft RCP	4-ft RCP
Eagle Drive	2-ft RCP	4-ft RCP
VICC1	2, 1.5-ft RCPs	2, 2x6 box
VICC2	2-ft CMP*	2, 2-ft RCP*
VICC3	2-ft RCP	2, 2x6 box
VICC4	1.5-ft RCP	2, 2x6 box
VICC5	1-ft PVC*	8' x 4' box*
Morningside 1	2, 2.5 and 1, 1.25 RCP	6x8 box
Morningside 2	3, 2.5-ft RCPs	6x8 box

Table 6-1. Proposed Culvert Improvements along RDLG
*culverts not entered in model

The cost of implementing all 10 culvert improvements is estimated at approximately $1.35 million and is further discussed in Section 6.3. These culverts were chosen based on the age of the culverts and the identification of drainage issues in these areas where it has been observed that flow through the culverts is impeded during high flow conditions.

The results of implementing Option 1 reveal very modest improvements in terms of damage assessments for the smaller frequency rainfall events and modest increases in damages for the larger frequency rainfall events. Figures 6-1 & 6-2 reveal the computed floodplains for the merged RDLG, NMD, and TR watershed area. It may be observed that at this scale virtually no difference may be seen when compared to the existing conditions scenario discussed in Section 4.2. Computed water surface elevations at the same locations discussed in Section 4.2 is presented in Tables 6-3 through 6-5. Again, it may be observed that very little variation is observed from the existing conditions scenario. For both NMD and the TR watersheds negligible differences are observed between existing conditions and the option 1 scenario. Throughout the RDLG watershed WSE's are actually seen increasing moving downstream towards the outfall. This is to be expected since the increased culvert capacity is moving stormwater flow towards the outfall at a faster rate thus increasing peak flow rates. However, when observing additional areas upstream, especially in the vicinity of Hanna High school near Shidler Rd. and Price Rd. on an offshoot of the Resaca, a lowering of predicted WSE's may be observed of nearly 0.5-ft (Table 6-6). This is an area that has historically been subjected to multiple flood events and has caused damages to some residential neighborhoods in that region. Furthermore, while WSE's were seen to increase slightly for the 10-yr event (under 0.3-ft), no negative impact was observed in terms of the overall floodplain or expected damages. In fact, a reduction in damages of over $1.5 million was calculated for the 10-yr frequency rainfall event largely attributed to decreased flood depths near Hanna High School. For the 100-yr rainfall event the implementation of the Option 1 culvert improvements alone resulted in increased flood depths of up to nearly 0.8-ft and an increase in expected damages of

	Option 1		
Water Depth (ft)	**2-Yr**	**10-Yr**	**100-Yr**
0-0.5	369	657	1383
0.5-1	220	442	987
1-1.5	101	329	695
1.5-2	38	194	497
2-2.5	18	75	309
2.5-3	6	33	159
3-3.5	10	13	71
3.5-4	2	8	40
4-4.5	1	10	21
4.5-5	2	9	11
5.0-8	3	16	31
8.0-12	0	1	2
>12.0	0	0	0
Total	770	1787	4206

Table 6-2. Number of Structures/Buildings predicted to be flooded in the merged watershed model (RDLG, NMD, and TR) under the Option 1 scenario for the 2-yr, 10-yr and 100-yr frequency rainfall events

nearly $300,000. The damage estimates were derived based on the data presented in Table 6-2 which summarizes the number of buildings within various flood depth ranges for the 2-yr, 10-yr and 100-yr storm events. While the overall variations between this scenario and the existing are minimal in terms of the number of buildings flooded, the benefits that are observed are due to flooded buildings experiencing shallower levels of inundation under the Option 1 scenario relative to the existing.

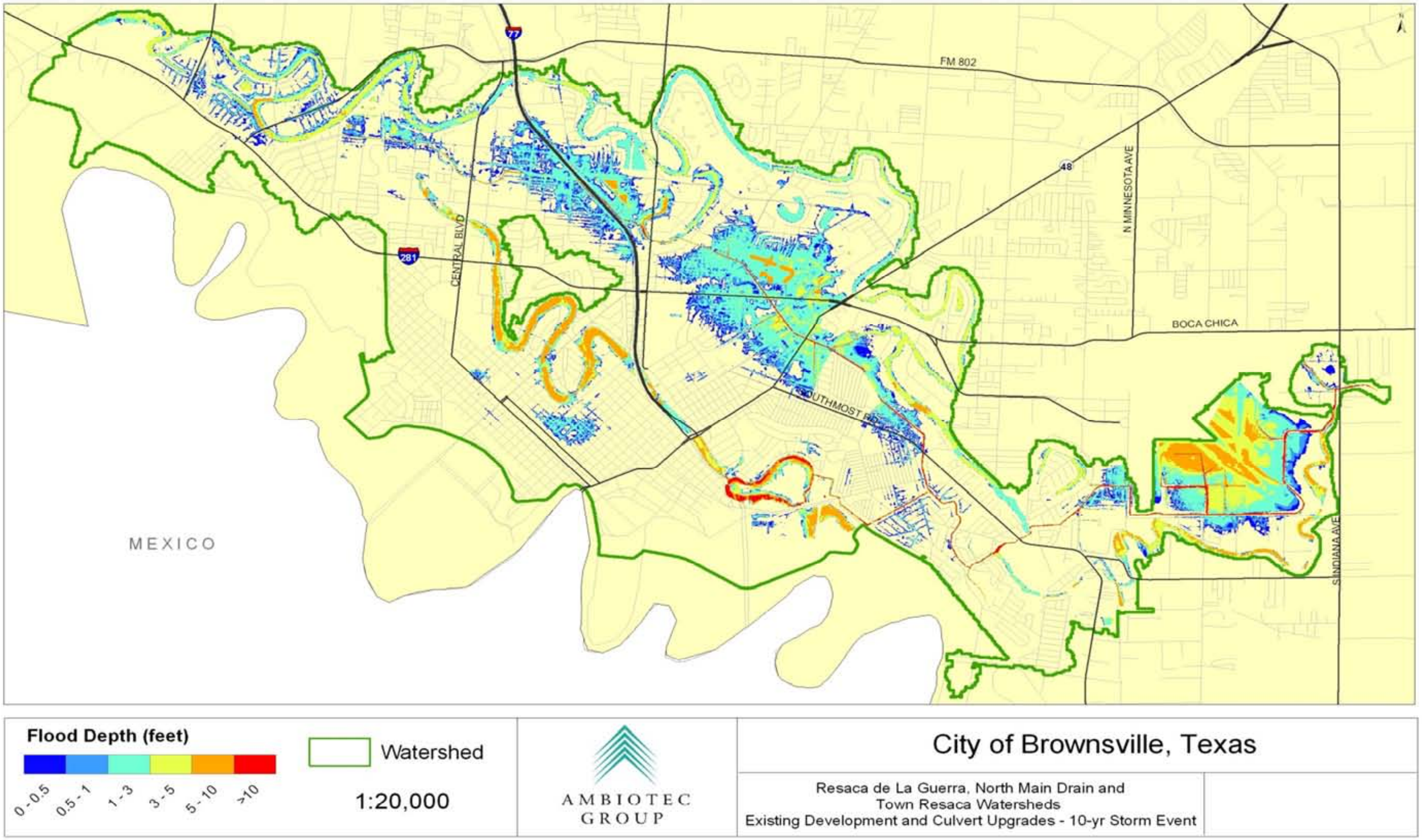

Figure 6-1. Option 1 floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

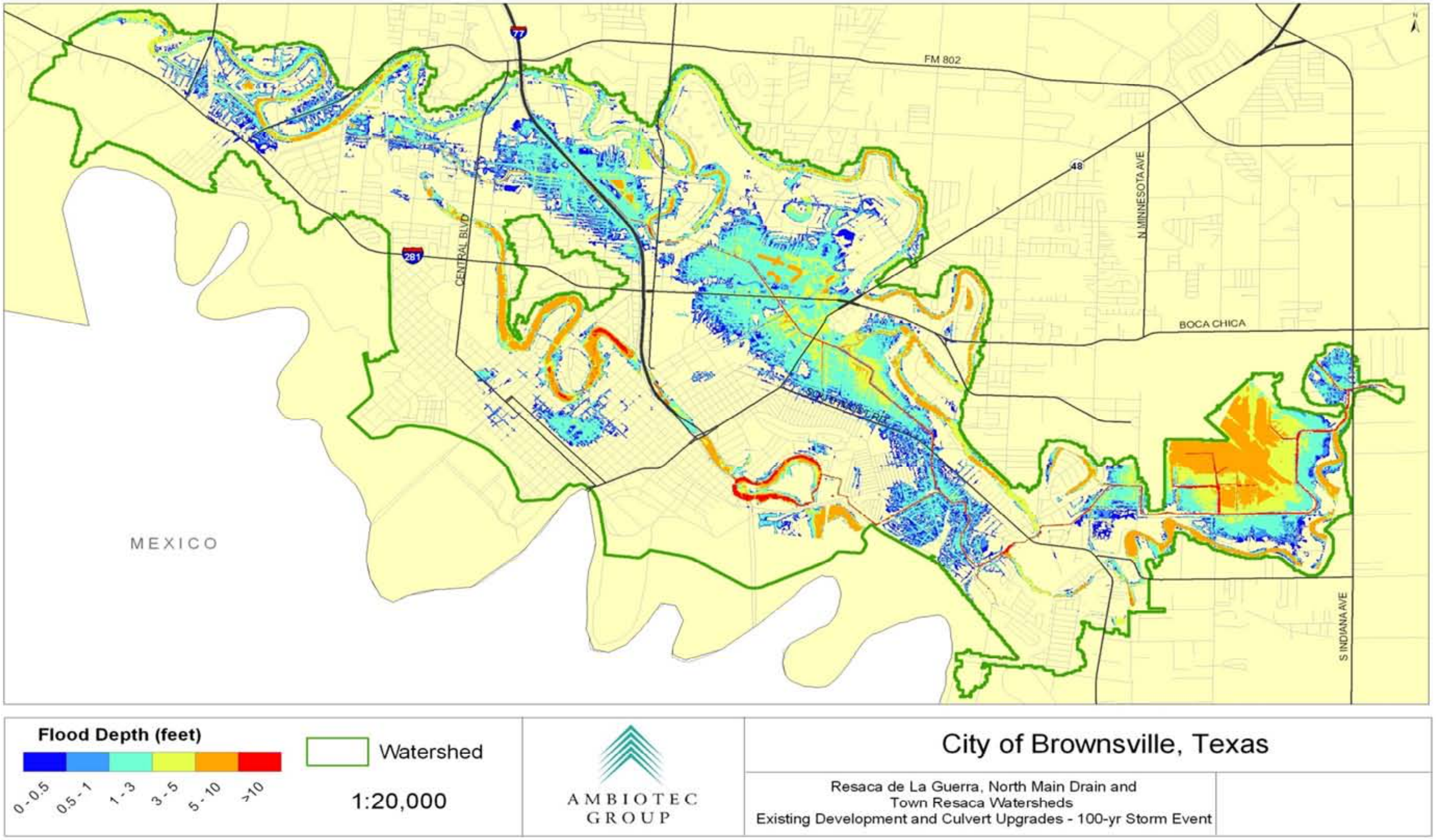

Figure 6-2. Option 1 floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. Option 1 (ft)
Alton Gloor	84480.30	10-yr	1.9	34.04	34.05
	84480.30	100-yr	15.3	34.42	34.43
Laredo Rd.	75741.20	10-yr	105.1	34.04	34.05
	75741.20	100-yr	162.5	34.42	34.42
Golf Course (upstream VICC)	67843.07	10-yr	647.3	33.24	33.24
	67843.07	100-yr	987.8	33.92	33.95
Central Blvd.	59913.97	10-yr	561.7	31.93	31.9
	59913.97	100-yr	990.3	32.53	32.54
US 77/83	58006.56	10-yr	561.7	31.51	31.49
	58006.56	100-yr	990.3	32.05	32.02
Old Alice	56431.98	10-yr	516.4	30.41	30.39
	56431.98	100-yr	1021.7	31.08	31.05
Paredes Line Rd.	49414.23	10-yr	413	29	28.98
	49414.23	100-yr	1017.8	30.12	30.11
Old Port Isabel	36395.52	10-yr	331.9	26.19	26.48
	36395.52	100-yr	944.6	28.59	28.59
Price Rd.	27465.97	10-yr	331.9	25.57	25.72
	27465.97	100-yr	944.6	27.07	27.1
Hwy 48	24796.52	10-yr	270.4	23.66	23.86
	24796.52	100-yr	896.3	26.06	26.19
Boca Chica	15543.40	10-yr	270.4	23.33	23.49
	15543.40	100-yr	896.3	24.9	25.07
Billy Mitchell Blvd.	13591.11	10-yr	258	22.36	23.14
	13591.11	100-yr	868.1	24.69	24.9
Acacia Lake Drive	11091.49	10-yr	258	22.32	23.12
	11091.49	100-yr	868.1	24.58	24.79
Morningside Rd.	483.63	10-yr	235.6	22.19	22.23
	483.63	100-yr	836.3	24.17	24.33
Outfall	55.00	10-yr	182.5	22.14	22.17
	55.00	100-yr	801.5	23.91	24.02

Table 6-3. RDLG Option 1 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. (ft)
Center	60774.16	10-yr	278	31.97	31.96
	60774.16	100-yr	420	32.25	32.25
Mesquite Grove Subdivision	56205.55	10-yr	556	30.74	30.71
	56205.55	100-yr	840	31.16	31.16
Hwy 77/83	48522.62	10-yr	1799.9	29.82	29.79
	48522.62	100-yr	2704.5	30.24	30.24
Old Port Isabel	43212.86	10-yr	1507.8	26.59	26.61
	43212.86	100-yr	2381.3	27.06	27.06
Boca Chica	40705.06	10-yr	1704.4	26.57	26.59
	40705.06	100-yr	2323.4	27.04	27.04
Btw Boca Chica and 14th	39911.47	10-yr	2666	26.57	26.58
	39911.47	100-yr	3742.4	27.04	27.04
Willow	31090.42	10-yr	2666	24.12	24.14
	31090.42	100-yr	3742.4	26.32	26.33
Esperanza	25254.22	10-yr	2209	22.83	22.86
	25254.22	100-yr	3508.3	24.92	25.01
La Posada	24392.11	10-yr	2209	22.66	22.69
	24392.11	100-yr	3508.3	24.86	24.96
Southmost	21375.80	10-yr	2875.6	21.08	21.09
	21375.80	100-yr	4303.1	22.11	22.11
Minnesota Ave.	15463.43	10-yr	2708.9	20.71	20.72
	15463.43	100-yr	3954.9	22.21	22.25
South Dakota	13087.34	10-yr	2625.7	20.91	20.92
	13087.34	100-yr	3820.7	22.5	22.55
East of Airport	6525.73	10-yr	1953.2	20.9	20.91
	6525.73	100-yr	2947.6	22.5	22.55
Utah	2744.40	10-yr	1943.7	20.69	20.7
	2744.40	100-yr	2938	22.29	22.34

Table 6-4. NMD Option 1 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. (ft)
Los Ebanos Blvd.	39377.80	10-yr	108	31.5	31.47
	39377.80	100-yr	161	31.87	31.83
Central Blvd.	37516.04	10-yr	180	31.49	31.45
	37516.04	100-yr	270	31.84	31.81
Boca Chica Blvd.	35034.97	10-yr	122	30.94	30.95
	35034.97	100-yr	243	31.76	31.75
Belthair St.	33604.69	10-yr	387	30.94	30.94
	33604.69	100-yr	582	31.75	31.75
Calle Retama	30282.79	10-yr	387	30.92	30.92
	30282.79	100-yr	582	31.73	31.72
Palm Blvd.	24605.86	10-yr	72	30.91	30.92
	24605.86	100-yr	207	31.72	31.71
Old Alice	22576.37	10-yr	222	30.9	30.91
	22576.37	100-yr	333	31.71	31.7
Ringgold St.	20190.48	10-yr	1217	29.98	29.98
	20190.48	100-yr	1837	31.01	31.01
US 77/83	16156.75	10-yr	878	29.92	29.93
	16156.75	100-yr	1473	30.92	30.92
12th St.	15339.09	10-yr	1051	29.85	29.86
	15339.09	100-yr	1778	30.78	30.78
International Blvd.	10907.08	10-yr	615	27.91	27.91
	10907.08	100-yr	615	28.62	28.62
East Ave.	4358.43	10-yr	1130	27.04	27.04
	4358.43	100-yr	1456	27.66	27.66
Calle Milpa Verde Dr.	1620.93	10-yr	1184	25.47	25.48
	1620.93	100-yr	1600	26.55	26.56
Outlet	242.56	10-yr	1201	24.2	24.21
	242.56	100-yr	1629	26.31	26.33

Table 6-5. TR Option 1 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. (ft)	Existing WSE - Option 1 WSE (ft)
RDLG offshoot near Shidler Rd.	3791.75	10-yr	11.5	28.95	0.45
	3791.75	100-yr	17	30.01	0.01
RDLG offshoot near Price Rd.	1596.08	10-yr	11.5	28.95	0.44
	1596.08	100-yr	17	30.01	0.01
RDLG offshoot near Eagle Drive	270.97	10-yr	115.1	28.93	0.02
	270.97	100-yr	170	30.01	0.01

Table 6-6. Computed Water Surface Elevations (WSE's) near Hanna High School

6.2 Inter-Watershed Flow Transfer from NMD to RDLG (Option 2)

The improvements for Option 2 include all of the culvert improvements discussed in Section 6.1 for Option 1 as well as a doubling of the pump capacity at the outfall of RDLG from approximately 53 cfs to 106 cfs and a 150 cfs diversion from NMD to RDLG along the Hike and Bike Trail ROW just west of Paredes Line Rd. The capital costs associated with the described improvements are estimated at $3,360,300 consisting of $2,239,050 for the flow diversion and $1,121,250 for the pump improvement. A further discussion of project costs including operation and maintenance costs and computed benefits will be presented in Section 6.3.

The overall impacts on flooding areas and depths upon implementation of Option 2 show mixed results. Review of the 10-yr and 100-yr floodplains that were delineated after implementation of this option (Figures 6-3 & 6-4) reveal no significant changes from those delineated under existing conditions. The water surface elevation data provided in Tables 6-7 through 6-9 indicate that water surface elevations increase throughout the tri-watershed area. While this result was expected throughout RDLG, especially downstream of the diversion from NMD, the overall impact to NMD was unknown prior to analysis. The numbers of flooded buildings associated with this scenario for the 10-yr and 100-yr event are approximately 1919 and 4414 respectively (Table 6-13). This can be compared to the 1790 and 4169 observed for each rainfall event under the existing condition scenario. This indicates for relatively large rainfall/stormflow events across the entire 3-watershed area, that diverting flow between watersheds would not be effective and could actually worsen conditions.

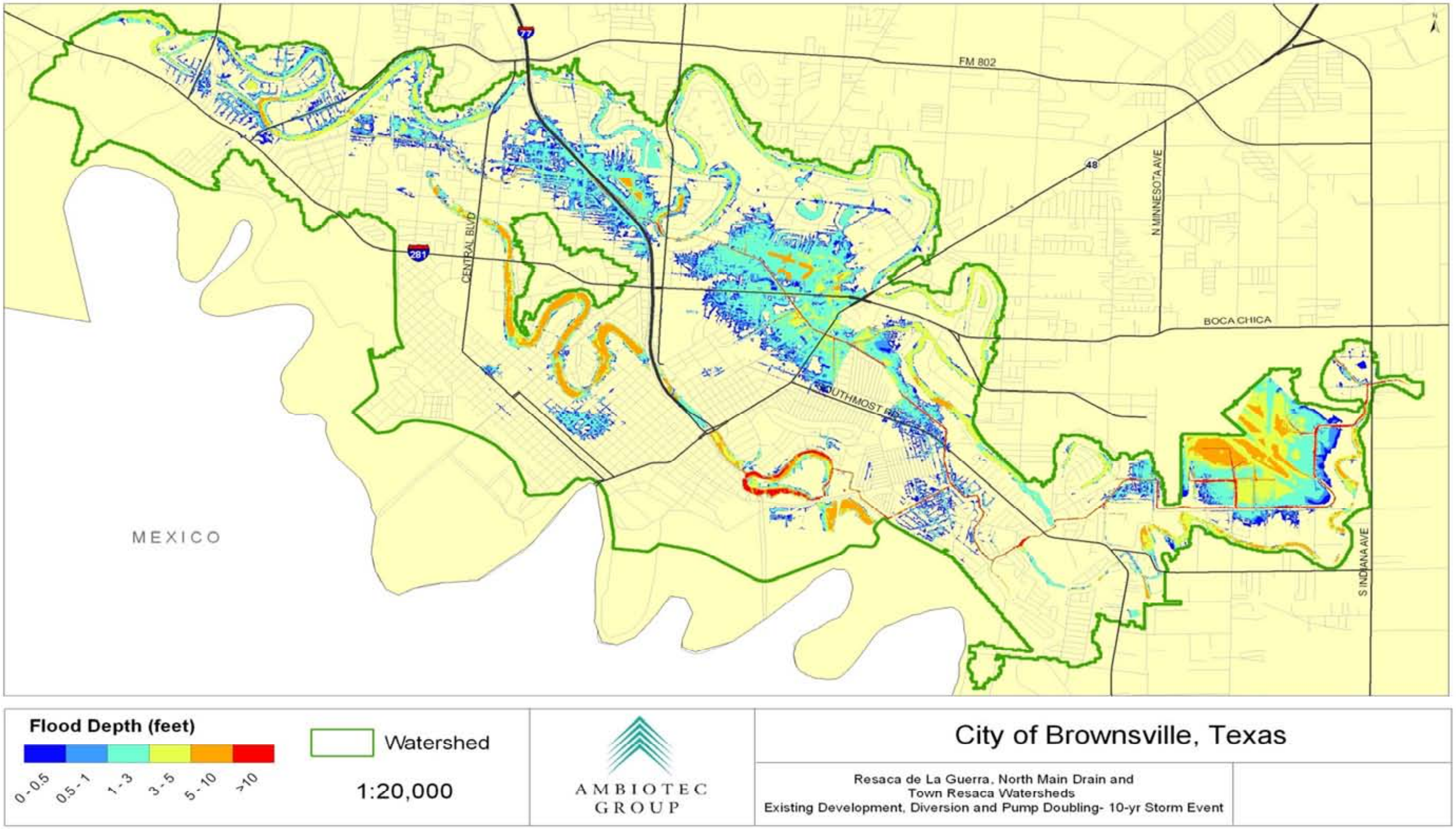

Figure 6-3. Option 2 floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 10-yr design rainfall total of 7.48 inches over a 24-hr period over the entire watershed area

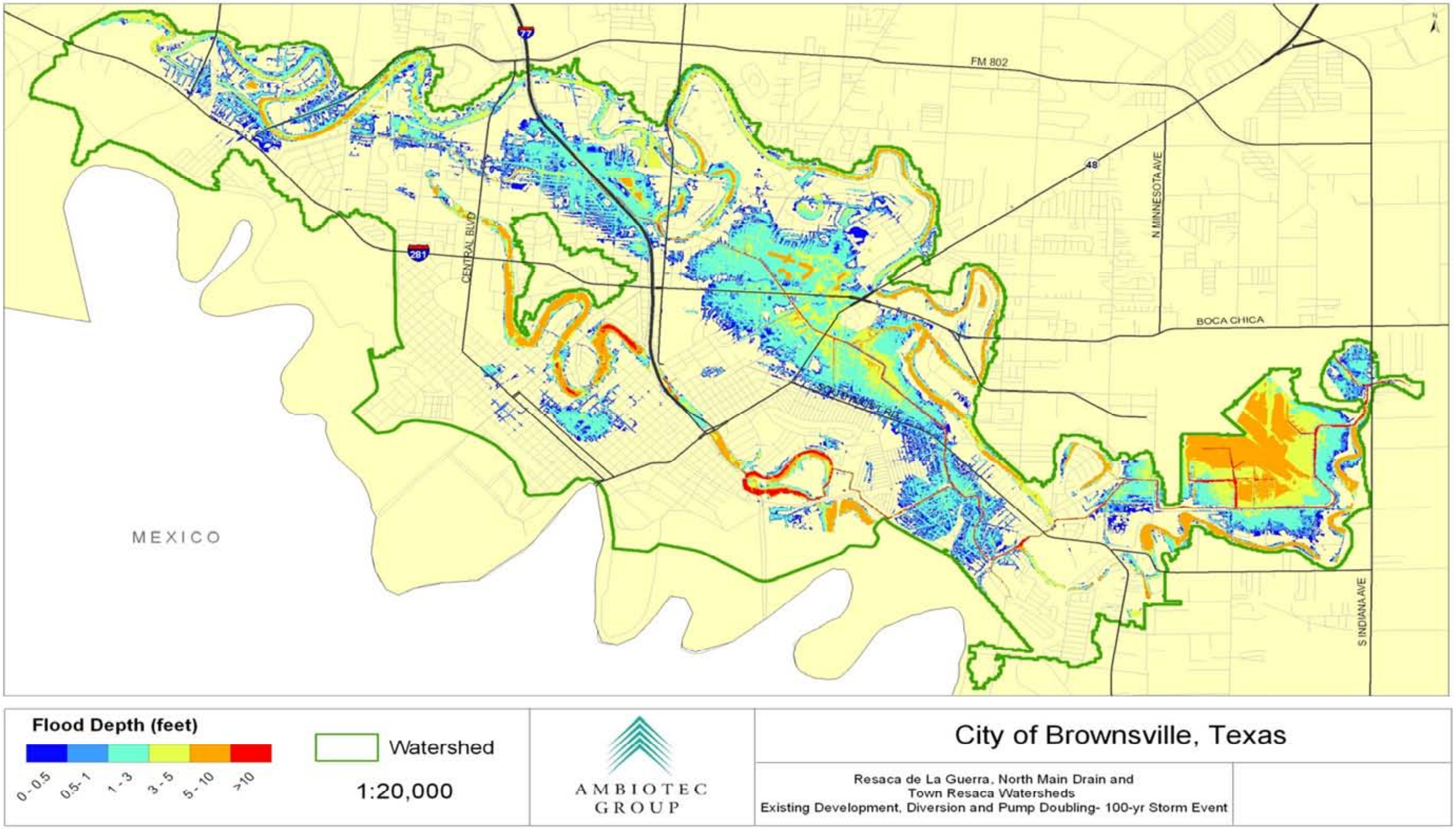

Figure 6-4. Option 2 floodplain for the merged model (RDLG, NMD, and TR watersheds) assuming a 100-yr design rainfall total of 11.75 inches over a 24-hr period over the entire watershed area

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. (ft)
Alton Gloor	84480.30	10-yr	1.9	34.04	34.05
	84480.30	100-yr	15.3	34.42	34.43
Laredo Rd.	75741.20	10-yr	105.1	34.04	34.05
	75741.20	100-yr	162.5	34.42	34.43
Golf Course (upstream VICC)	67843.07	10-yr	647.3	33.24	33.24
	67843.07	100-yr	987.8	33.92	33.95
Central Blvd.	59913.97	10-yr	561.7	31.93	31.89
	59913.97	100-yr	990.3	32.53	32.51
US 77/83	58006.56	10-yr	561.7	31.51	31.49
	58006.56	100-yr	990.3	32.05	32.04
Old Alice	56431.98	10-yr	516.4	30.41	30.4
	56431.98	100-yr	1021.7	31.08	31.15
Paredes Line Rd.	49414.23	10-yr	549.9	29	29.28
	49414.23	100-yr	1167.8	30.12	30.44
Old Port Isabel	36395.52	10-yr	474.8	26.19	27.35
	36395.52	100-yr	1105.3	28.59	28.83
Price Rd.	27465.97	10-yr	474.8	25.57	26.29
	27465.97	100-yr	1105.3	27.07	27.37
Hwy 48	24796.52	10-yr	422.1	23.66	24.51
	24796.52	100-yr	1096.3	26.06	26.6
Boca Chica	15543.40	10-yr	422.1	23.33	23.81
	15543.40	100-yr	1096.3	24.9	25.32
Billy Mitchell Blvd.	13591.11	10-yr	411.7	22.36	23.54
	13591.11	100-yr	1076.7	24.69	25.17
Acacia Lake Drive	11091.49	10-yr	411.7	22.32	23.49
	11091.49	100-yr	1076.7	24.58	25.02
Morningside Rd.	483.63	10-yr	350.7	22.19	22.13
	483.63	100-yr	1054.5	24.17	24.38
Outfall	55.00	10-yr	242.6	22.14	22.01
	55.00	100-yr	973.3	23.91	23.92

Table 6-7. RDLG Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. (ft)
Center	60774.16	10-yr	278	31.97	31.96
	60774.16	100-yr	420	32.25	32.25
Mesquite Grove Subdivision	56205.55	10-yr	556	30.74	30.73
	56205.55	100-yr	840	31.16	31.16
Hwy 77/83	48522.62	10-yr	1799.9	29.82	29.82
	48522.62	100-yr	2704.5	30.24	30.24
Old Port Isabel	43212.86	10-yr	1419.4	26.59	26.63
	43212.86	100-yr	2170.9	27.06	27.11
Boca Chica	40705.06	10-yr	1628.5	26.57	26.61
	40705.06	100-yr	2275.1	27.04	27.09
Btw Boca Chica and 14th	39911.47	10-yr	2603.6	26.57	26.61
	39911.47	100-yr	3696.4	27.04	27.09
Willow	31090.42	10-yr	2603.6	24.12	24.93
	31090.42	100-yr	3696.4	26.32	26.63
Esperanza	25254.22	10-yr	2672.3	22.83	23.06
	25254.22	100-yr	3987.1	24.92	25.18
La Posada	24392.11	10-yr	2672.3	22.66	22.77
	24392.11	100-yr	3987.1	24.86	25.12
Southmost	21375.80	10-yr	2740.4	21.08	21
	21375.80	100-yr	4194.9	22.11	22.1
Minnesota Ave.	15463.43	10-yr	2601.1	20.71	20.6
	15463.43	100-yr	3915.3	22.21	22.18
South Dakota	13087.34	10-yr	2520.3	20.91	20.79
	13087.34	100-yr	3808.2	22.5	22.48
East of Airport	6525.73	10-yr	1880.3	20.9	20.78
	6525.73	100-yr	2885.2	22.5	22.48
Utah	2744.40	10-yr	1871.7	20.69	20.57
	2744.40	100-yr	2875.9	22.29	22.27

Table 6-8. NMD Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total (cfs)	W.S.E. Existing (ft)	W.S.E. (ft)
Los Ebanos Blvd.	39377.80	10-yr	108	31.5	31.52
	39377.80	100-yr	161	31.87	31.82
Central Blvd.	37516.04	10-yr	180	31.49	31.51
	37516.04	100-yr	270	31.84	31.8
Boca Chica Blvd.	35034.97	10-yr	122	30.94	30.96
	35034.97	100-yr	243	31.76	31.76
Belthair St.	33604.69	10-yr	387	30.94	30.96
	33604.69	100-yr	582	31.75	31.75
Calle Retama	30282.79	10-yr	387	30.92	30.94
	30282.79	100-yr	582	31.73	31.73
Palm Blvd.	24605.86	10-yr	72	30.91	30.93
	24605.86	100-yr	207	31.72	31.72
Old Alice	22576.37	10-yr	222	30.9	30.92
	22576.37	100-yr	333	31.71	31.71
Ringgold St.	20190.48	10-yr	1217	29.98	30
	20190.48	100-yr	1837	31.01	31.01
US 77/83	16156.75	10-yr	878	29.92	29.95
	16156.75	100-yr	1473	30.92	30.92
12th St.	15339.09	10-yr	1051	29.85	29.88
	15339.09	100-yr	1778	30.78	30.78
International Blvd.	10907.08	10-yr	615	27.91	27.95
	10907.08	100-yr	615	28.62	28.63
East Ave.	4358.43	10-yr	1130	27.04	27.12
	4358.43	100-yr	1456	27.66	27.68
Calle Milpa Verde Dr.	1620.93	10-yr	1184	25.47	25.71
	1620.93	100-yr	1600	26.55	26.75
Outlet	242.56	10-yr	1201	24.2	24.92
	242.56	100-yr	1629	26.31	26.6

Table 6-9. TR Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E)

While the larger rainfall events illustrated undesirable results for the flow transfer from NMD to RDLG, the 2-yr rainfall event illustrated modest improvements. Tables 6-10 through 6-12 list computed water surface elevations throughout the three watersheds relative to that of existing conditions. While water surface elevations in RDLG are still elevated relative to existing conditions due to the increased flow rates, values for NMD are observed to slightly drop between Old Port Isabel and the area between Boca Chica and 14th Street. While these reductions are very minor the damage assessment indicated a benefit of over $725,500 for the 2-yr rainfall event. The number of computed flooded buildings for the 2-yr, 10-yr and 100-yr rainfall frequency events relative to the existing conditions scenario may be observed in Table 6-13. The overall benefit-cost analysis of this scenario considering the probability of having any given frequency rainfall event over the planning period will be discussed in Section 6.3

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total Existing(cfs)	Q Total Option 2 (cfs)	W.S.E. Existing(ft)	W.S.E. Option 2(ft)
Alton Gloor	84480.30	2-yr	2.1	1	33.61	33.57
Laredo Rd.	75741.20	2-yr	63.2	63.2	33.61	33.57
Golf Course (upstream VICC)	67843.07	2-yr	401.9	397.7	32.83	32.79
Central Blvd.	59913.97	2-yr	226.5	224.1	31.24	31.24
US 77/83	58006.56	2-yr	226.5	224.1	30.79	30.76
Old Alice	56431.98	2-yr	195	193.2	28.89	29.86
Paredes Line Rd.	49414.23	2-yr	166.5	215.5	28.36	28.64
Old Port Isabel	36395.52	2-yr	144.6	230.2	24.95	25.59
Price Rd.	27465.97	2-yr	144.6	230.2	24.69	25.07
Hwy 48	24796.52	2-yr	116.9	162.5	22.98	23.15
Boca Chica	15543.40	2-yr	116.9	162.5	22.87	22.95
Billy Mitchell Blvd.	13591.11	2-yr	102.1	151.3	21.34	21.54
Acacia Lake Drive	11091.49	2-yr	102.1	151.3	21.33	21.51
Morningside Rd.	483.63	2-yr	100.5	150.8	20.84	20.83
Outfall	55.00	2-yr	39.4	45	20.82	20.77

Table 6-10. RDLG Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E) for the 2-yr design storm

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total Existing(cfs)	Q Total Option 2 (cfs)	W.S.E. Existing(ft)	W.S.E. Option 2(ft)
Center	60774.16	2-yr	150	150	31.69	31.69
Mesquite Grove Subdivision	56205.55	2-yr	300	300	30.14	30.13
Hwy 77/83	48522.62	2-yr	1008.4	1008.4	28.98	28.97
Old Port Isabel	43212.86	2-yr	946.8	864.7	25.7	25.6
Boca Chica	40705.06	2-yr	1080.7	1023	25.67	25.56
Btw Boca Chica and 14th	39911.47	2-yr	1591.4	1514.5	25.66	25.56
Willow	31090.42	2-yr	1591.4	1514.5	23.36	23.34
Esperanza	25254.22	2-yr	1995.8	1987.1	21.76	21.72
La Posada	24392.11	2-yr	1995.8	1987.1	21.44	21.39
Southmost	21375.80	2-yr	2038.5	2025.8	20.12	20.07
Minnesota Ave.	15463.43	2-yr	1810.6	1773.3	19.61	19.55
South Dakota	13087.34	2-yr	1705.2	1672	19.71	19.65
East of Airport	6525.73	2-yr	1376.1	1348	19.7	19.64
Utah	2744.40	2-yr	1377	1349.4	19.55	19.49

Table 6-11. NMD Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E) for the 2-yr design storm

Location	Cross-Section (HEC-RAS)	Rainfall Frequency	Q Total Existing(cfs)	Q Total Option 2 (cfs)	W.S.E. Existing(ft)	W.S.E. Option 2(ft)
Los Ebanos Blvd.	39377.80	2-yr	60	60	31.27	31.27
Central Blvd.	37516.04	2-yr	100	100	31.25	31.25
Boca Chica Blvd.	35034.97	2-yr	57	57	28.71	28.71
Belthair St.	33604.69	2-yr	217	217	28.7	28.7
Calle Retama	30282.79	2-yr	217	217	28.34	28.34
Palm Blvd.	24605.86	2-yr	26	26	28.33	28.32
Old Alice	22576.37	2-yr	123	123	28.31	28.3
Ringgold St.	20190.48	2-yr	705	705	27.58	27.57
US 77/83	16156.75	2-yr	460	460	27.53	27.52
12th St.	15339.09	2-yr	552	552	27.43	27.43
International Blvd.	10907.08	2-yr	543	543	26.61	26.61
East Ave.	4358.43	2-yr	786	786	25.73	25.72
Calle Milpa Verde Dr.	1620.93	2-yr	800	800	24.58	24.57
Outlet	242.56	2-yr	810	810	23.34	23.31

Table 6-12. TR Option 2 Conditions Flows (Q Total) and Predicted Water Surface Elevations (W.S.E) for the 2-yr design storm

	Option 2		
Water Depth (ft)	**2-Yr**	**10-Yr**	**100-Yr**
0-0.5	355	757	1364
0.5-1	189	446	1076
1-1.5	86	338	735
1.5-2	34	203	553
2-2.5	16	84	329
2.5-3	7	34	171
3-3.5	10	13	73
3.5-4	1	8	45
4-4.5	1	9	22
4.5-5	2	10	13
5.0-8	3	16	31
8.0-12	0	1	2
>12.0	0	0	0
Total	704	1919	4414

Table 6-13. Number of Structures/Buildings predicted to be flooded in the merged watershed model (RDLG, NMD, and TR) under the Option 2 scenario for the 2-yr, 10-yr and 100-yr frequency rainfall events

6.3 Recommendations / Updated Capital Improvement Plan (CIP)

The 2006 Flood Protection Plan included a series of structural and non-structural strategies to mitigate flooding issues throughout the City. The structural improvements that required a capital cost to implement were prioritized based on a cost-benefit analysis and presented in a 20-yr Capital Improvement Plan (CIP) broken down into 4, 5-yr phases. The non-structural improvements were also discussed but since they required little capital to implement relative to the other recommended improvements, were given high priority and recommended for immediate implementation. This section re-visits the 2006 CIP to update projects in terms of cost, continued feasibility and completion of projects since the 2006 study was adopted by the City. Additionally, the benefit-cost analysis for those projects simulated during this study (Option 1 and Option 2) will be presented and added to the overall CIP as appropriate.

6.3.1 Benefit-Cost Analysis for Option 1 and Option 2 for Merged Model (RDLG, NMD, TR)

To assess the overall effectiveness in reducing flood damages from a Benefit-Cost perspective, both Option 1 and Option 2 were analyzed using the methodology developed in the 2006 Flood Study. 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 existing scenario over a twenty-year (20-yr) planning horizon and a forty-year (40-yr) project life. These computed benefits were then divided by the overall capital costs associated with the construction, engineering, and land acquisition of each option to arrive at a ratio of benefits to costs. Using this methodology, any values greater than one (1) have benefits that exceed the costs and are generally viewed positively and those less than one (1) have costs that exceed benefits and are determined to be inefficient from a benefit-cost perspective. Furthermore, the greater the number is above one (1) the greater the overall benefits relative to the cost and the higher priority that project should be given.

The benefit-cost (B/C) analysis was completed for the Merged Model Option 1 discussed in Section 6.1. The overall capital costs associated with all culvert improvements modeled in Option 1 was approximately $1,350,000. When considering the net present value (NPV) of this cost over the 40 year project life and considering operation and maintenance costs, the NPV of the total cost comes to approximately $3,381,250. The NPV of the total benefits of this scenario amounts to $4,242,547 leading to an overall benefit-cost ratio of 1.25. This value indicates the cost-efficiency of the project even though the damages were observed to slightly increase for the larger rainfall frequency events (50-yr and 100-yr). The cause of the increase in damages for the higher frequency storms is the faster conveyance of water through the system due to the increased culvert sizes. While the increased culvert sizes help move water through the system more quickly and alleviate flooding in upstream areas, the backwater effect at the outfall is increased causing increased flooding further downstream. However, since the smaller frequency rainfall events have a much higher probability of occurring in any given year, the benefits provided for storms smaller than a 50-yr outweigh the additional damages observed for larger rainfall events. Table 6-14 summarizes the overall NPV of Total Costs and Benefits along with the B/C Ratio for Option 1.

As discussed in Section 6.2, Option 2 included all of the culvert improvements from Option 1 in addition to a diversion of approximately 150 cfs from NMD to RDLG along the existing Hike and Bike Trail ROW along with a pump improvement at the outfall of RDLG increasing its capacity from 53 cfs to 106 cfs. The overall capital costs for the improvements in option 2, not including the culvert improvements from option 1, are approximately $3,360,300. The overall capital cost for the improvements discussed in Option 1 and Option 2 together total approximately $4,710,300. The NPV of this cost over a 40 year project life and including operation and maintenance costs come to $8,612,675. This compares to an overall benefit of implementation of $9,585,462 arriving at a B/C ratio of 1.11. Similar to the analysis completed for Option 1, the overall expected damages for each frequency event revealed a benefit for the small rainfall totals and an increase in damages for the larger rainfall events. However, in this case it was only the 2-yr rainfall event where a benefit was observed. For all other larger rainfall totals, damages were actually observed to increase. Table 6-14 summarizes the NPV of Total Costs and Benefits along with the B/C Ratio for Option 2 (which includes the improvements from Option 1).

	Option 1	**Option 1 & 2**
NPV Total Costs	$ 3,381,250	$ 8,612,675
NPV Total Benefits	$ 4,242,547	$ 9,585,462
B/C Ratio	1.25	1.11

Table 6-14. Benefit-Cost comparisons for implementation of Options 1 & 2

6.3.2 Projects Completed Since the 2006 CIP

Since the completion of the 2006 Flood Protection Plan and its adoption by the City, several projects from the recommended CIP have been implemented throughout the City. These projects include:

- Phase I of the Towne North Detention Pond (Approximately half of the full proposed storage capacity) – CCDD1
- Owen's Road Detention is currently under design – NMD
- The culvert over CCDD1 at Highway 48 was replaced with a bridge structure
- A detention pond / water park has been designed at the CCDD1 site
- Additional storage capacity has been created at the Brownsville Country Club through the creation of additional Resaca-type features that may be drained prior to rainfall events (approximately 10 ac-ft with plans to construct another 10-11 ac-ft of storage)

Other culvert improvements that have been made that were not explicitly recommended in the 2006 CIP but have improved overall drainage conditions include:

- Culvert improvements at the three Laredo Rd. Crossings
- Culvert improvement at Hidden Valley Rd.
- Culvert improvement at Palo Verde Rd.

In addition to these projects from the 2006 study, design is currently under way for several of the culvert improvements along RDLG that were proposed in Option 1 of the Merged Model analysis. The specific culvert improvements currently under way include 5 culverts within Valley International Country Club (VICC), the Eagle Drive culvert, and the two Morningside Rd. crossings.

Finally, there has been much progress in recent years with regards to coordination and cooperation activities amongst local drainage entities. One example of this was during Hurricane Dolly in 2008. Prior to landfall, representatives from local entities including the City of Brownsville, Cameron County Drainage District No. 1 and Brownsville Irrigation District all worked together to pump down permanent pool water elevations throughout the local resacas. This in effect created extra storage capacity during the storm event and minimized flooding impacts. While it is difficult to quantify the exact dollar impact of this activity, it was noted that there were multiple areas throughout the City that are typically subjected to flood waters during large rainfall events, that were not impacted or impacted as much during Hurricane Dolly.

6.3.3 Capital Improvement Plan

This section of the report re-examines the CIP and schedule recommended in the 2006 Flood Protection Plan to include the new projects discussed in this report, remove projects that have either already been completed (discussed in Section 6.3.2) or are no longer feasible (due to land availability or some other technical limitation), and updates costs based on current 2011 estimates. Because the Benefit-Cost analysis was already completed in the 2006 and projects were selected based on both cost efficiency and effectiveness in reducing flood damages, the analysis will not be redone here. While costs for each project have been updated to reflect current market costs for the sake of the CIP, it is assumed that land and home values would also have risen since the 2006 study and that projects with a high B/C ratio then, would still have a high B/C ratio now. These observations combined with the high B/C ratios that were observed during the 2006 analysis, provided sufficient evidence that higher costs today would not alter the bottom line of the Benefit-Cost analysis.

The overall analysis results from the 2006 study are presented in Table 6-15. This table shows the NPV of Total Costs, Benefits, B/C Ratio and Net Benefits for all projects in each of the four watersheds that were analyzed. In the case of RDLG, no projects analyzed resulted in a B/C ratio greater than one (1) and as such no projects were recommended in the plan for that watershed. The capital costs of all projects proposed in the 2006 study used in the analysis presented in Table 6-15 was updated as presented in Tables 6-16 through 6-18. Additionally, Table 6-19 presents the total costs

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

Table 6-15. Cost Benefit Analysis from 2006 Flood Protection Plan

recommended from the analysis of the merged model in this present study. Based on the benefit-cost analysis discussed in Section 6.3.1, the culvert improvements analyzed in Option 1 were added as a proposed improvement due it's overall cost-efficiency in mitigating flood damages despite the fact that damages were increased for both the 50-yr and 100-yr storm. While it is never desirable to alleviate flooding conditions in an upstream area at the expense of the downstream area in this situation, the worsening of flooding in the downstream area for large rainfall events could be alleviated through the implementation of the pump capacity expansion project discussed in Option 2 in addition to continued planning and coordination efforts amongst entities regulating water levels in the RDLG system as discussed in Section 6.3.2. While the flow diversion from NMD to RDLG that was also analyzed in Option 2 still showed a positive B/C ratio, a negative impact was witnessed for rainfall events as small as a 5-yr frequency which are much more common in the region. As such, a permanent flow diversion between watersheds cannot be recommended and will not be included in this CIP.

CCDD1 Proposed Improvements	Costs
Implement Runoff Controls for New Developments	N/A
Remove and Replace Weir Structure @ Paredes Line Road	$365,700
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
Construct Dana Road Detention Ponds (2)	$19,507,000
Construct Towne North Detention Pond (half completed)	$4,259,250
Construct Robindale Road Detention Pond	$14,289,200
Construct FM 802 Detention Ponds (2)	$40,566,500
Construct Detention Pond on Minnesota and Austin Road	$7,357,400
Construct Detention Pond on Minnesota near Airport	$8,321,000
Replace FM 3248 (Alton Gloor) culvert with 3, 10'x10' box culverts	$1,690,500
Replace Paredes Line Road culvert with 3, 10'x10' box culverts	$1,690,500
Replace Old Port Isabel Road culvert with 3, 10'x10' box culverts	$1,621,500
Replace FM 802 culvert with 3, 10'x10' box culverts	$1,776,750
CCDD No. 1 Ditch No. 1 Improvement Totals:	**$101,823,300**

Table 6-16. Updated **e**stimated construction costs of the proposed CCDD1 improvements from the 2006 Flood Protection Plan

North Main Drain Improvements	Costs
Construct Price Road Detention Pond	$1,200,600
Construct City Detention Pond Near Owens Road (Under Design)	
Construct City Detention Pond Near Airport	$7,486,500
Construct Levee around southern portion of airport	$1,029,600
Line ditch to top of bank from 77/83 to RDLG confluence	$16,900,000
Line ditch to top of bank from RDLG confluence to airport	$4,225,000
North Main Drain Improvement Totals:	**$30,841,700**

Table 6-17. Updated **e**stimated construction costs of the proposed NMD improvements from the 2006 Flood Protection Plan

Town Resaca Proposed Improvements	Costs
Impala Pump Station Upgrade	$1,621,500
Dredge Town Resaca near Brownsville Zoo	$6,565,000
Line Ditch from South WWTP to Impala Pump Station	$2,047,500
Property Buyouts	$950,000
TOWN RESACA IMPROVEMENT TOTALS:	**$11,184,000**

Table 6-18. Updated **e**stimated construction costs of the proposed TR improvements from the 2006 Flood Protection Plan

Merged Model - RDLG Proposed Improvements	Costs
Option 1	
10 culvert improvements throughout RDLG	$1,350,000
Option 1 Totals	$1,350,000
Option 2	
Pump Improvement at outfall of RDLG	$1,121,250
Option 2 Totals	$1,121,250
MERGED MODEL IMPROVEMENT TOTALS:	**$2,471,250**

Table 6-19. Estimated construction costs of the proposed Merged Model improvements from the 2011 CIP Flood Protection Plan Update

Overall, the projects recommended from the 2006 analysis along with the culvert improvements and increased pump capacity at the outfall of RDLG analyzed in this analysis, total over $146 million in capital improvement costs. This would mean that an investment of approximately $7.3 million/year over a 20-yr period would need to be made to fully implement the projects proposed in this analysis. Clearly, this will present a significant financial burden to the City and local drainage entities and provides clear evidence of the need for the capture of external funding sources as well as a cooperative approach between local and regional drainage entities to protect the City and southern Cameron County from flood damages. While not examined in this analysis, another important consideration is the potential loss of life of local citizens that could result from major flood events. Flooding is considered the most frequent and costly of all natural disasters in the United States (NWF, 1998) and as such should be made a priority in terms of dedicating resources towards mitigating.

The 2006 Flood Protection Plan CIP proposed a series of 4, 5-yr CIPs to diffuse the overall project costs over the 20-yr planning period. This concept is maintained and presented again in Tables 6-21 through 6-23 including updated project costs, additional projects, and the removal of completed or now infeasible projects. Each 5-yr period CIP includes investments totaling between $30 and over $45 million. This proposed CIP is not fixed and stone and may be adjusted based on funding availability and requirements, changing development patterns, regulatory scenarios and financial priorities through the use of the hydrologic and hydraulic models that were developed as part of this project. The proposed CIP phases, including a description of the projects, associated capital costs and potential funding sources are also provided in Tables 6-21 through 6-23. 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.

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Construct Price Road Detention Pond	$1,200,600	B
Complete Design and Construction of City Detention Pond Near Owens Road (Currently Under Design)		
Construct City Detention Pond Near Airport	$7,486,500	B
Construct levee around southern portion of Airport	$1,029,600	B
Total NMD:	**$9,716,700**	

****CIP I continued on following page

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	$365,700	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	$2,125,000	B
Purchase Land for Robindale Road Detention Pond	$1,325,000	B
Purchase Land for FM 802 Detention Pond	$4,250,000	B
Complete Remaining Portion of Towne North Detention Pond	$4,259,250	B
Purchase Land for Minnesota & Austin Road Detention Pond	$650,000	B
Total CCDD1:	**$13,352,950**	

Town Resaca		
Proposed Improvement	Estimated Costs	Funding
Property Buyouts	$950,000	F
Impala Pump Station Upgrade	$1,621,500	B
Line Ditch from South WWTP to Impala Pump Station	$2,047,500	C
Total TR:	**$4,619,000**	

Resaca de la Guerra		
Proposed Improvement	Estimated Costs	Funding
Culvert Improvement at 5 VICC Culverts	$675,000	
Culvert Improvement at Upstream Morningside Rd. Crossing	$135,000	
Culvert Improvement at Downstream Morningside Rd. Crossing	$135,000	
Culvert Improvement at Shidler Rd.	$135,000	
Culvert Improvement at Price Rd.	$135,000	
Culvert Improvement at Eagle Drive	$135,000	
Pump Improvement at Outfall of RDLG	$1,121,250	
Total RDLG:	**$2,471,250**	
Total Costs:	**$30,159,900**	

Table 6-20. Updated Phase I CIP (Years 1-5)

North Main Drain		
Proposed Improvement	Estimated Costs	Funding
Line ditch to top of bank from 77/83 to RDLG confluence	$16,900,000	B
Line ditch to top of bank from RDLG confluence to Airport	$4,225,000	B
Total NMD:	**$21,125,000**	

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
		B
Construct Detention Pond on Minnesota and Austin Road	$6,707,400	B
Construct Dana Road Detention Ponds (2)	$17,382,000	B
Total CCDD1:	**$24,089,400**	
Total Costs:	**$45,214,400**	

Table 6-21. Updated Phase II CIP (Years 6-10)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Replace FM 802 culvert with 3, 10'x10' box culverts	$1,776,750	B
Replace Old Port Isabel Rd. culvert with 3, 10' x 10' boxes	$1,621,500	B
Construct Detention Pond on Minnesota near Airport	$8,321,000	B
Replace Paredes Line Rd. culvert with 3, 10' x 10' boxes	$1,690,500	B
Construct Robindale Road Detention Pond	$12,964,200	B
Total CCDD1:	**$26,373,950**	

Town Resaca		
Proposed Improvement	Estimated Costs	Possible Funding Source
Dredge Town Resaca near Brownsville Zoo	$6,565,000	CO
Total TR:	**$6,565,000**	
Total Costs:	**$32,938,950**	

Table 6-22. Updated Phase III CIP (Years 11-15)

CCDD No. 1		
Proposed Improvement	Estimated Costs	Possible Funding Source
Construct FM 802 Detention Ponds (2)	$36,316,500	B
Replace FM 3248 (Alton Gloor) culvert with 3, 10' x 10' boxes	$1,690,500	B
Total Costs:		**$38,007,000**

Table 6-23. Updated Phase IV CIP (Years 16-20)

6.3.4 Non-Structural Recommendations/Additional Planning Elements

In addition to the proposed capital improvements discussed in Section 6.3.2, 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, Cameron County and the Brownsville Irrigation District among others. These overlapping jurisdictions cross watershed boundaries, diffuse authority and accountability, and make it difficult to develop and implement consistent and cost effective drainage strategies and policies within 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.

Install Streamflow and Rainfall Gaging Network – Streamflow data is critical to the development of representative hydraulic and hydrologic models. Since the 2006 study was completed a SCADA system has been installed throughout some Brownsville resacas that record stage and allow drainage and irrigation entities manage water levels. However, there is still a need for flow gages throughout all resacas and drainage ditches in addition to rainfall gages so that rainfall totals can be correlated to flow and water levels throughout the drainage system. 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.

Development of a Flood Alert System – The storm surge analysis that was completed in this phase II study reveal a large portion of the City that could under significant levels of water if the area were to be impacted by a large hurricane. Due to the regions susceptibility to storm surge because of the City's proximity to the coast, it is infeasible to construct any type of project that could completely protect the City from such an event. As such, providing adequate prediction and lead time for evacuation activities becomes of utmost importance. For this reason through the use of the streamflow and rainfall gaging network proposed above in conjunction with radar rainfall prediction capabilities, the hydrologic and hydraulic models developed for the area and web browser capabilities, it is recommended that an alert system be considered for the region.

Continued Coordination and Cooperation Between Regulatory Entities - The effectiveness of coordination and cooperation activities amongst local drainage entities including the City of Brownsville, the Cameron County Drainage District No. 1 and the Brownsville Irrigation District as discussed in Section 6.3.2 was illustrated in 2008 with the onset of Hurricane Dolly. It is of critical importance that these types of activities continue in the future to aid in the mitigation of flood damages throughout the City especially as long as there are still multiple entities with regulatory authority over drainage features throughout the City.

6.3.5 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.

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TWDB Comments

1. Please include an Executive Summary for the Final Report.

   Executive summary added to report; pgs. ii – xiv

2. Section 1.3; on Page 4, the number of culverts surveyed as part of the field data collection activities has been left blank. Please amend.

   This has been addressed in the referenced section. The number of culverts surveyed for the project was in excess of 70.

3. Section 2.2.11; on Page 18, the table number referenced in the text has been left blank. Please amend

   This has been amended; the section now refers to table 2.2.11

4. Throughout the report, numerous references are made to sections of the report that do not exist. For example:
   - Section 2.1.5; the last sentence references a discussion in Section 3.4, but there is not a Section 3.4.
   - Section 2.2.11; several references are made throughout this section to discussions and information presented in Section 2.4, but there is no Section 2.4.
   - Section 3.1.1; reference is made to Section 3.3 and Section 3.4 that do not exist.

   Please amend and perform a thorough review to ensure there are no other references to incorrect sections elsewhere in the report.

   A thorough review of the entire document was made and all of the aforementioned errors were corrected in addition to a couple of other Sections that were incorrectly referenced.

5. Figure 3-2 illustrates the inundation depths associated with existing development and a 10-yr storm event, while Figure 3-3 shows depth of flooding under existing development and the 100-yr storm event. Comparing the two figures seems to indicate that flooding depths are greater under the 10-yr event then the 100-yr event, particularly east of Hwy 77. Please explain.

   The incorrect image was inserted into Figure 3-2 and instead of showing the 10-yr floodplain under existing conditions it was showing the 10-yr floodplain under maximum hurricane storm surge conditions. This error has been corrected and all figures now display what is described in the text/captions.

6. In comparing the depths of flooding indicated by Figure 3-2 to Figure 5-1, which illustrates flood depths under the same development and storm event scenarios but adds the maximum hurricane storm surge, the two figures appear to be identical. Please explain why storm surge would have no, or minimal, impact to depth of flooding.

   See explanation in 5. This has been corrected.

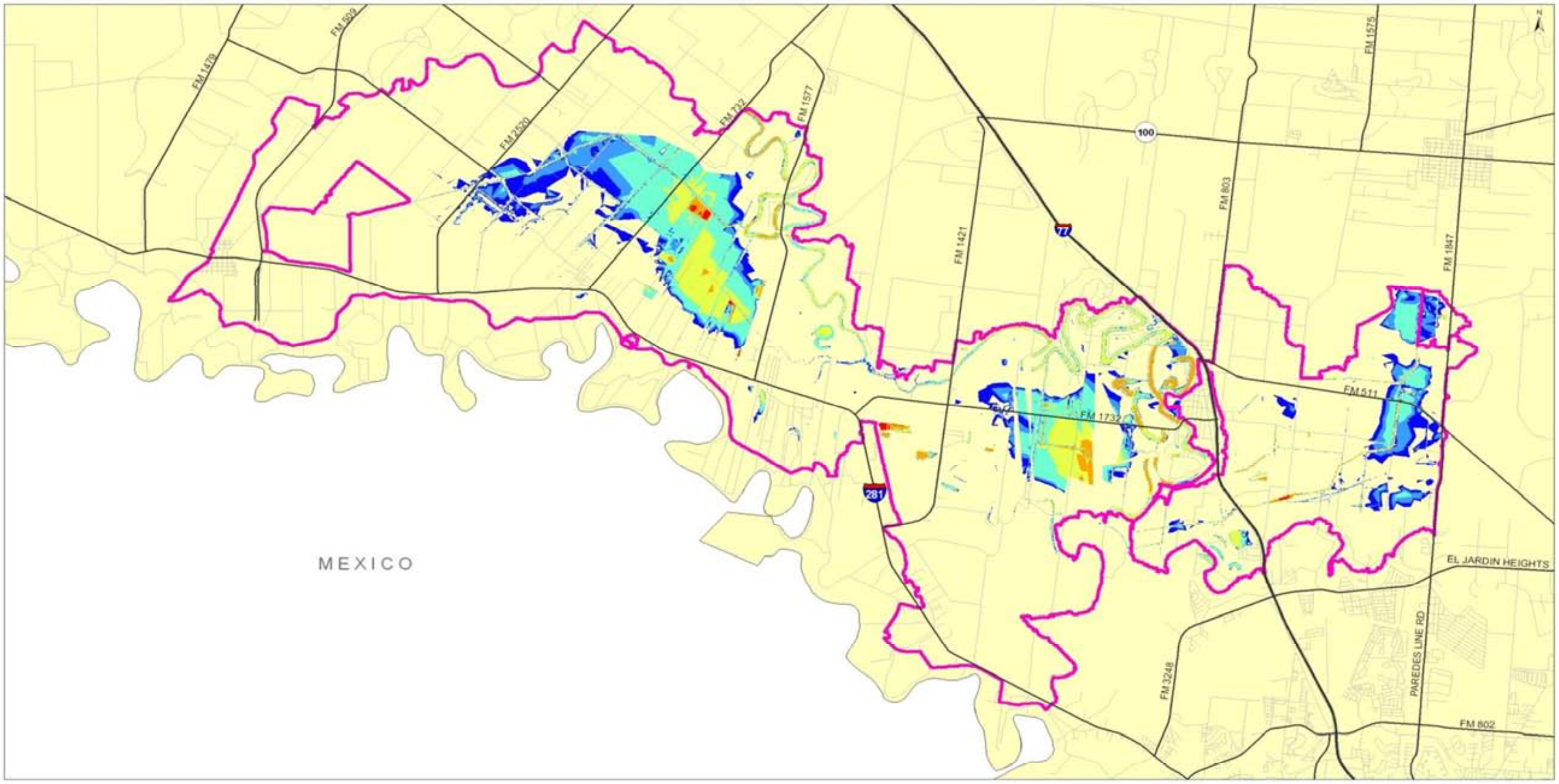
7. The study follows standard methodologies and practice utilizing acceptable HEC modeling in the engineering aspects of hydrologic and hydraulic techniques. The hydrologic modeling parameters were determined based on the calculation and engineering judgements for the existing and ultimate conditions. Mitigation alternatives identified by the study are eligible for funding under the Board's financial assistance programs. Application requirements and eligibility criteria is identified by Board rules specified in Section 363 of the Texas Administrative Code. The report would be appropriate for use in support of an application to the Board for financing the proposed improvements. All additional information required by Board rules, 31 TAC 363.401-404, as well as necessary information to make legal findings as required by Texas Water Code Chapter 17.771-776, would be required at the time of loan application.

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX A



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

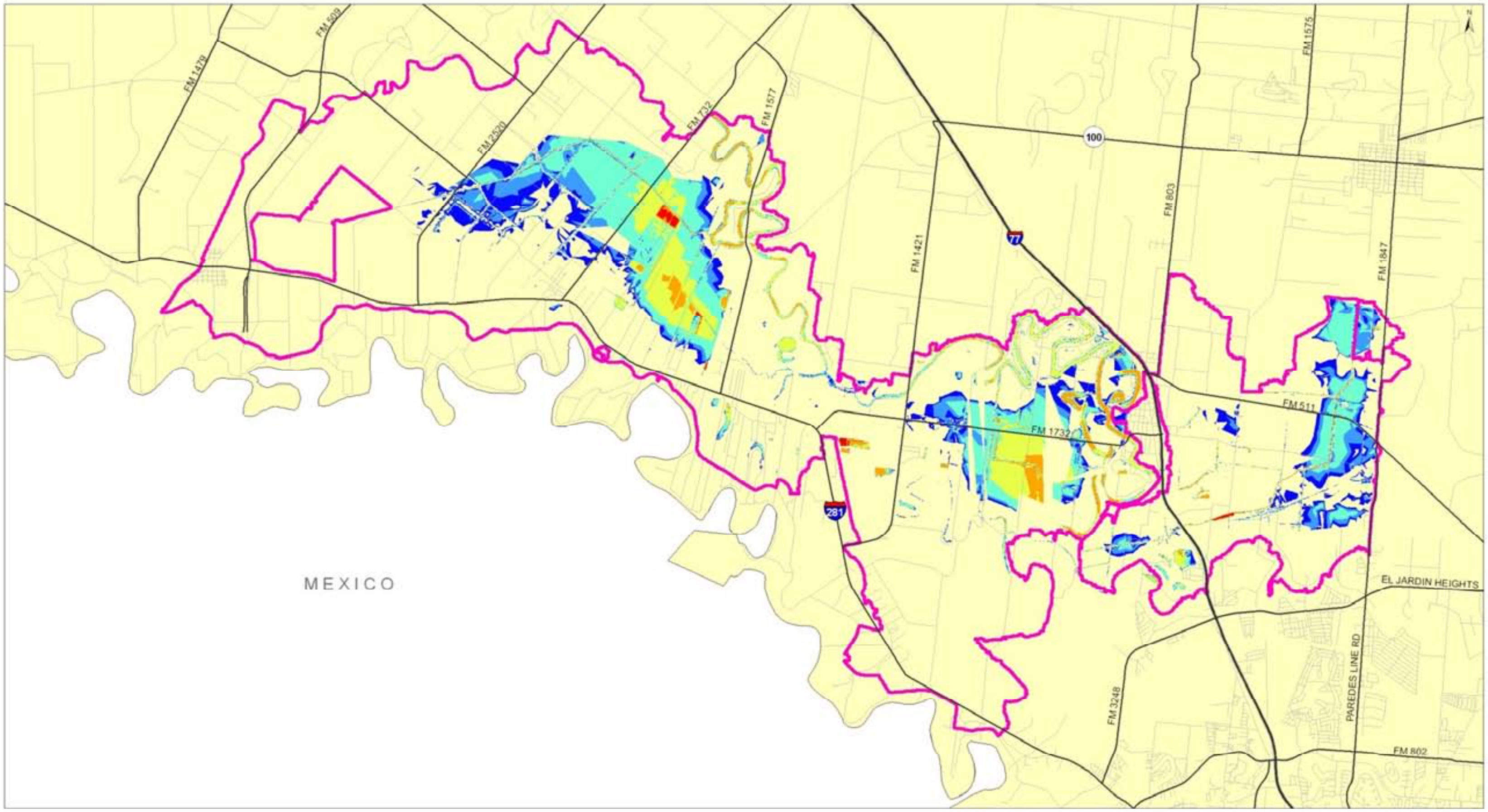
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development - 2-yr Storm Event



Flood Depth (feet)

0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10

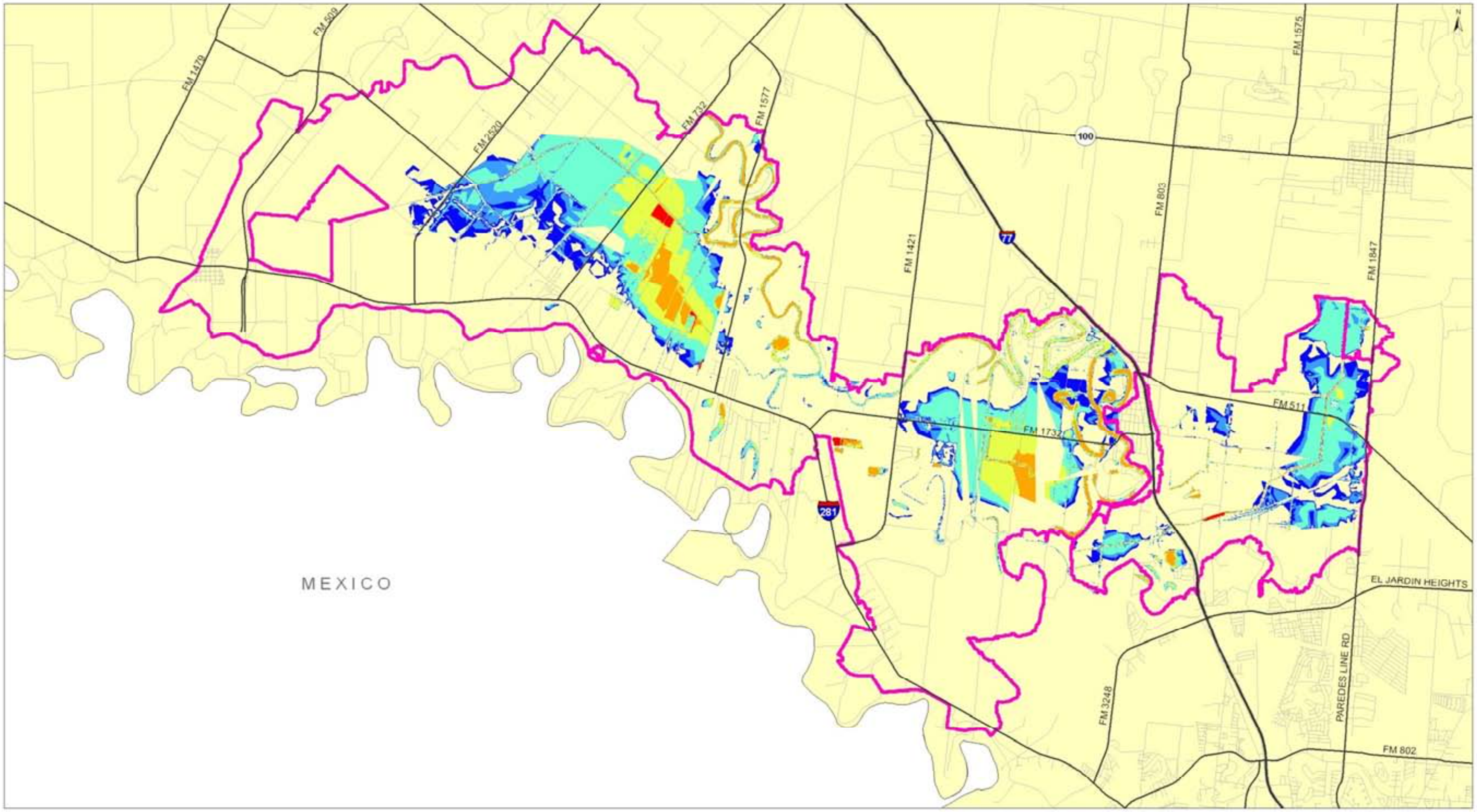
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

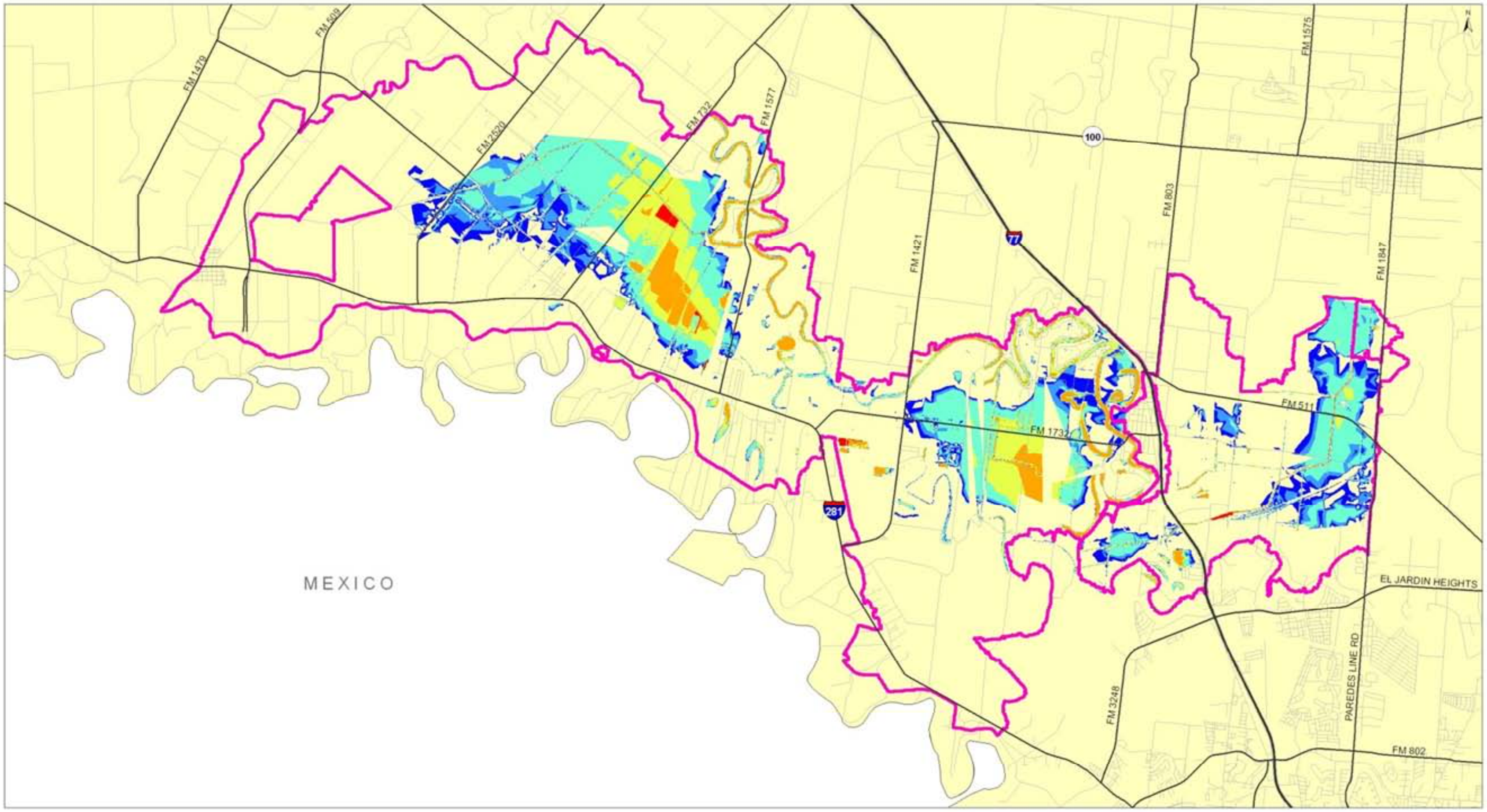
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

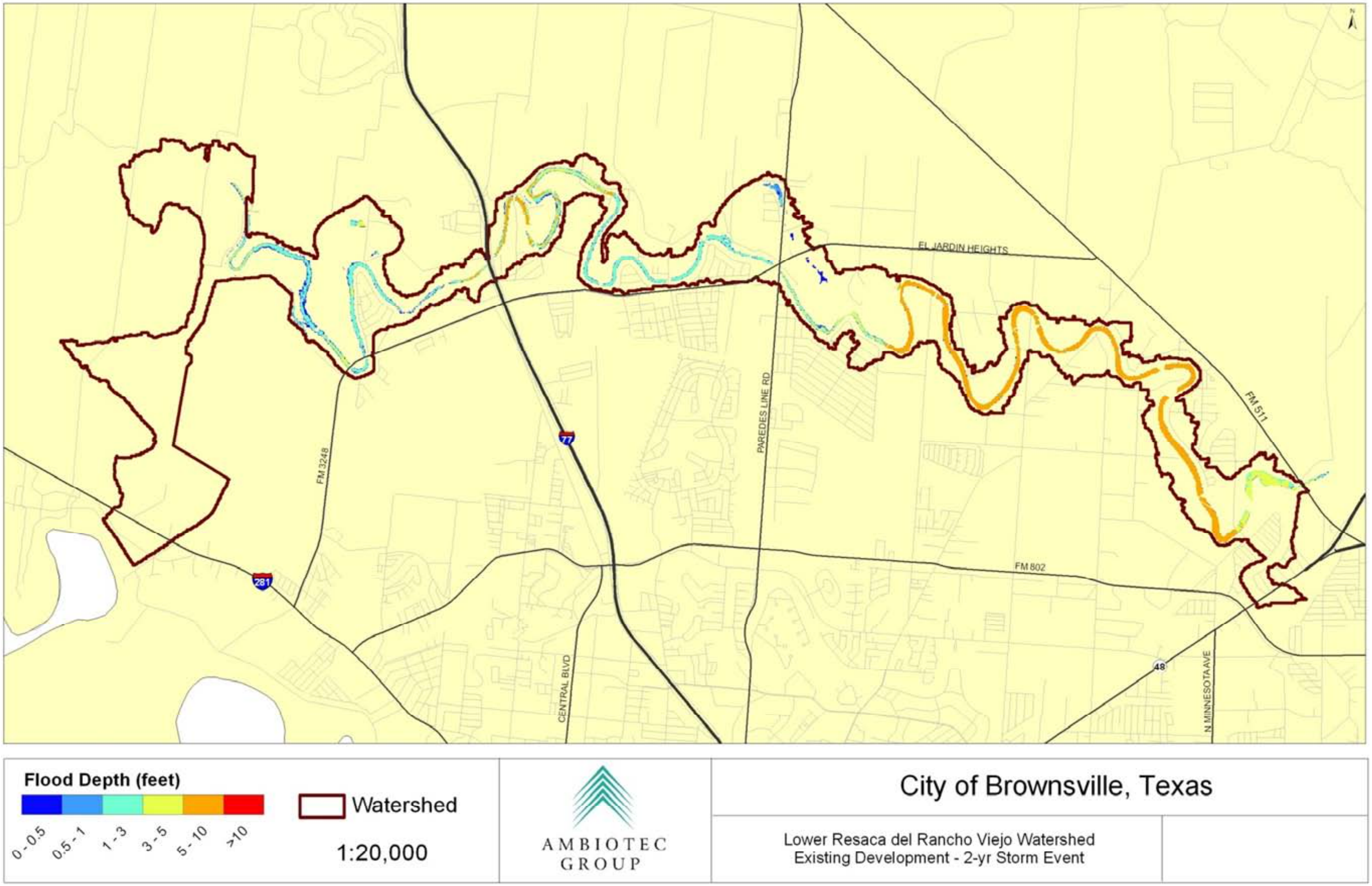
Watershed

1:40,000

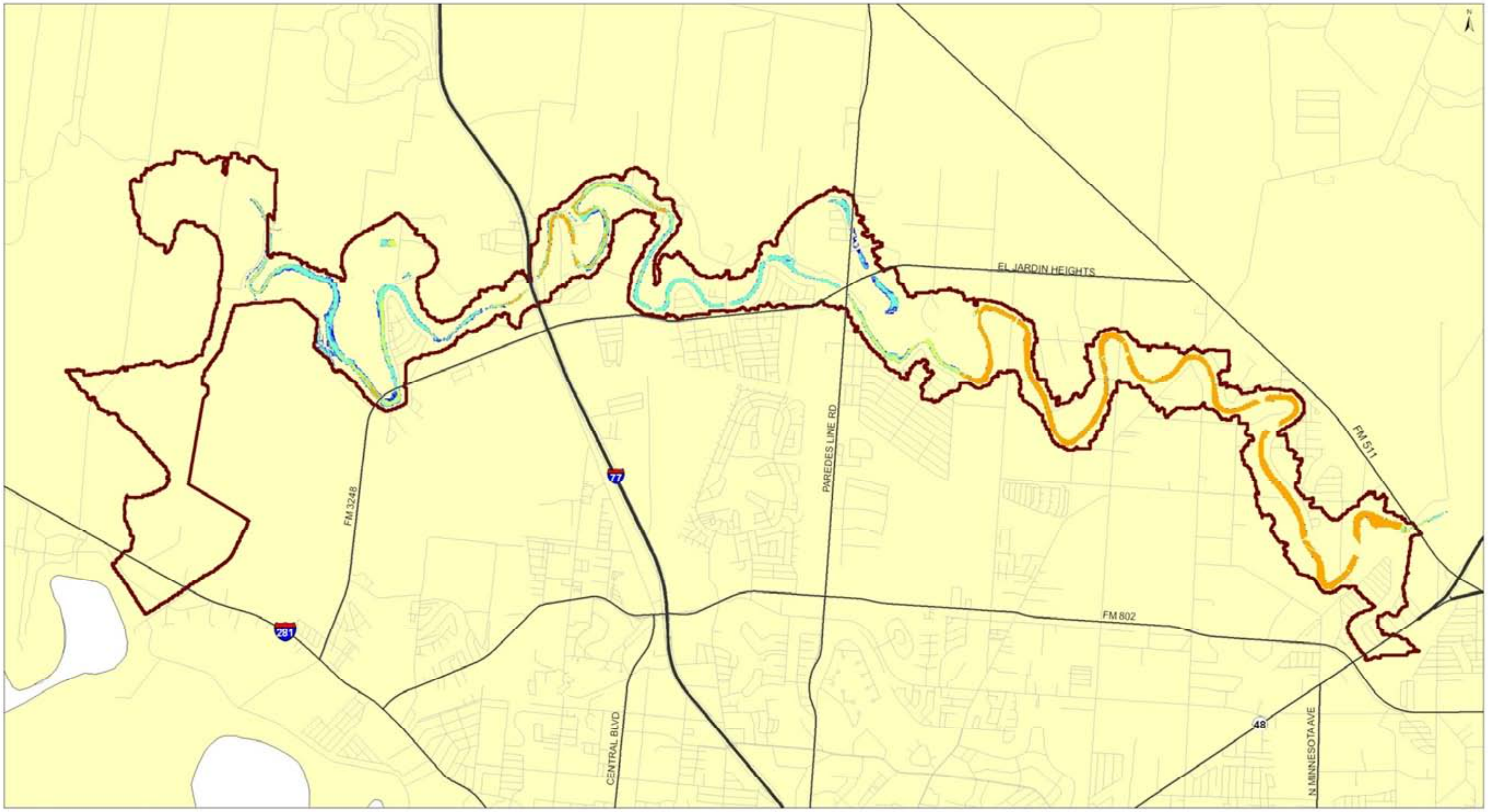
AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development - 50-yr Storm Event



EL JARDIN HEIGHTS
FM 511
FM 3248
PAREDES LINE RD
FM 802
CENTRAL BLVD
N MINNESOTA AVE
77
281
48
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:20,000
AMBIOTEC GROUP
City of Brownsville, Texas
Lower Resaca del Rancho Viejo Watershed
Existing Development - 2-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

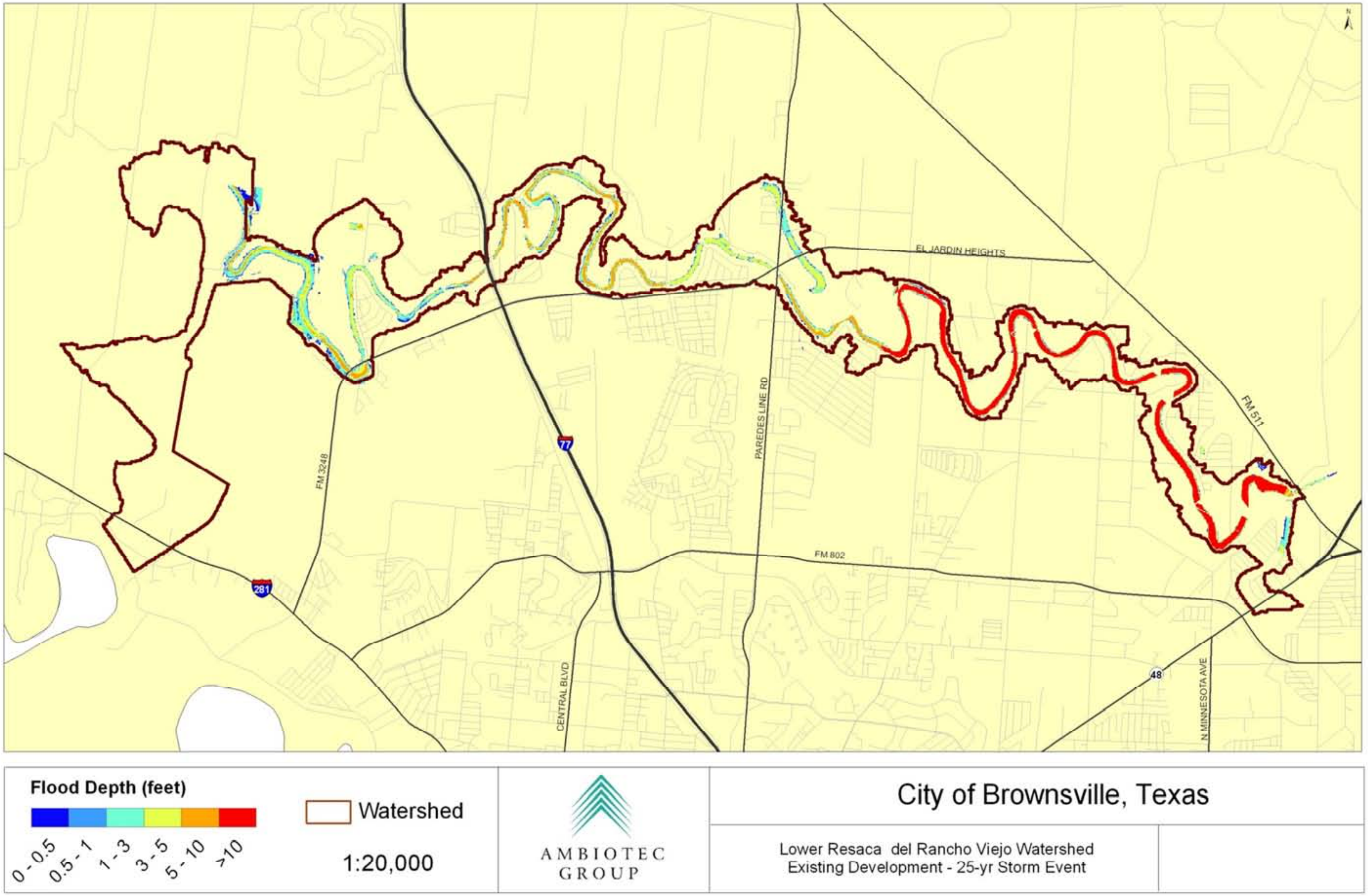
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Existing Development - 5-yr Storm Event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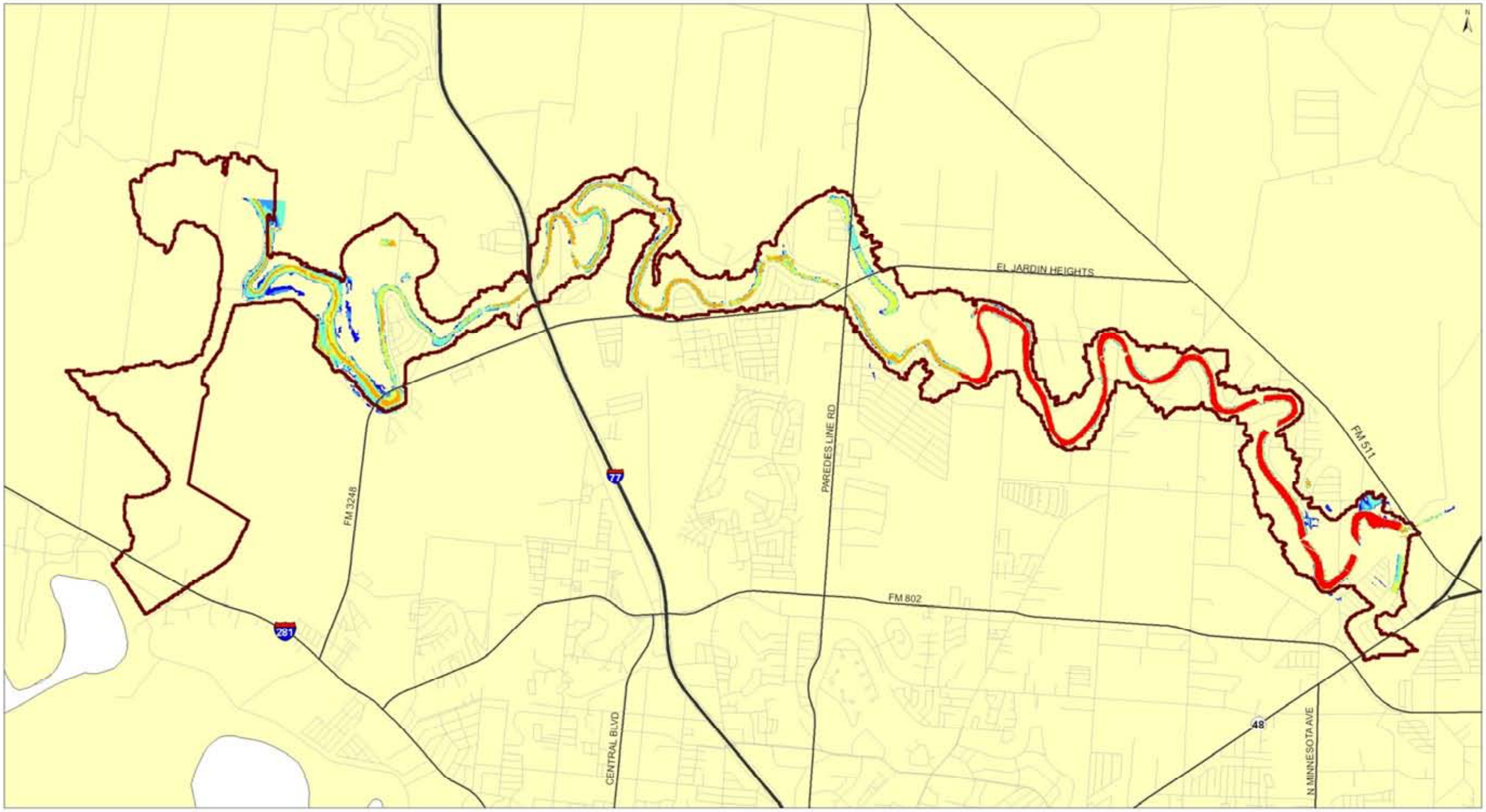
FM 511
PAREDES LINE RD
FM 3248
77
FM 802
281
CENTRAL BLVD
48
N MINNESOTA AVE
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:20,000
AMBIOTEC
GROUP
City of Brownsville, Texas
Lower Resaca del Rancho Viejo Watershed
Existing Development - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

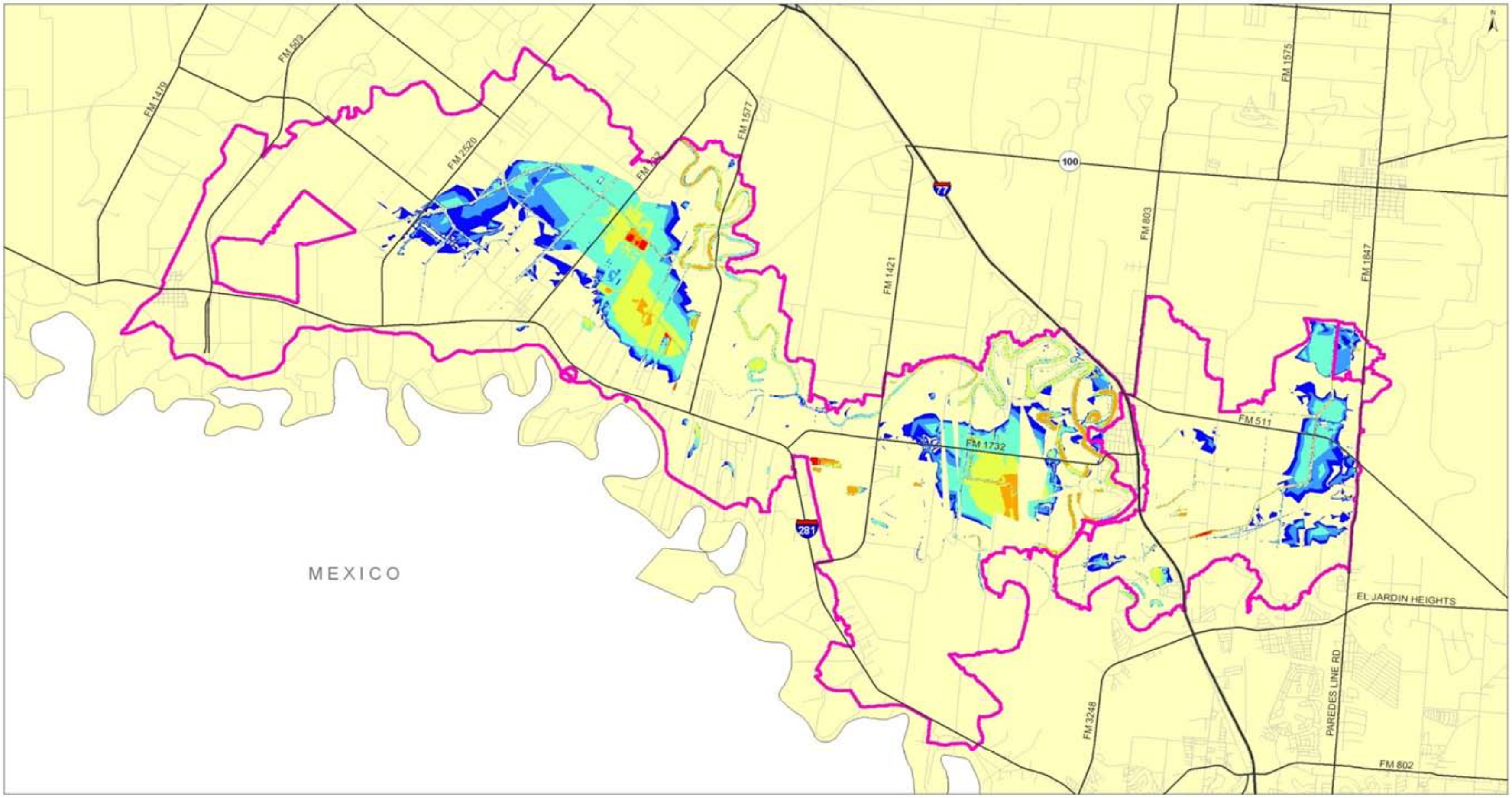
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Existing Development - 50-yr Storm Event



Flood Depth (feet)

0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10

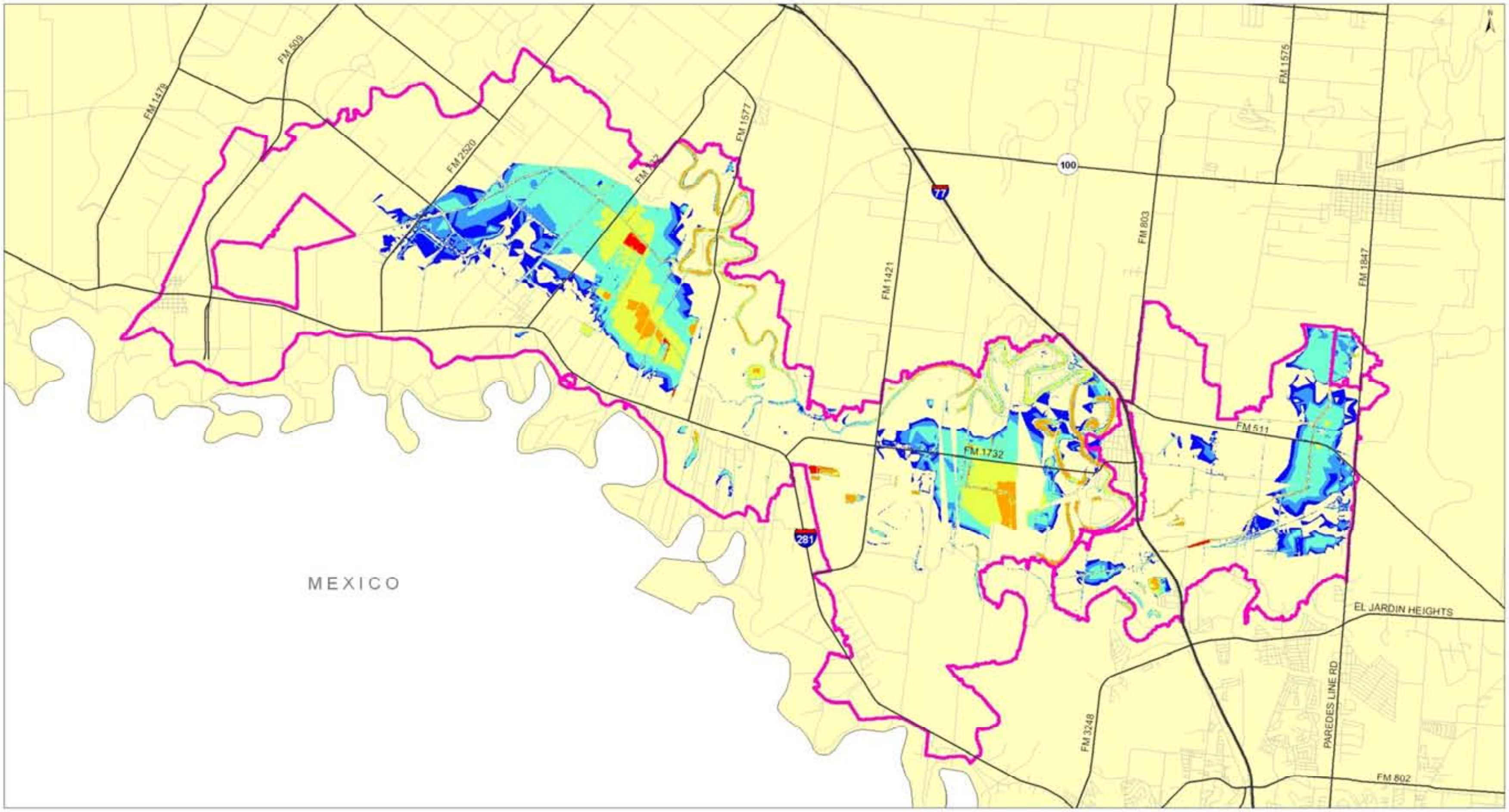
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3 and
Upper Resaca Rancho Viejo Watersheds
Full Development - 2-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

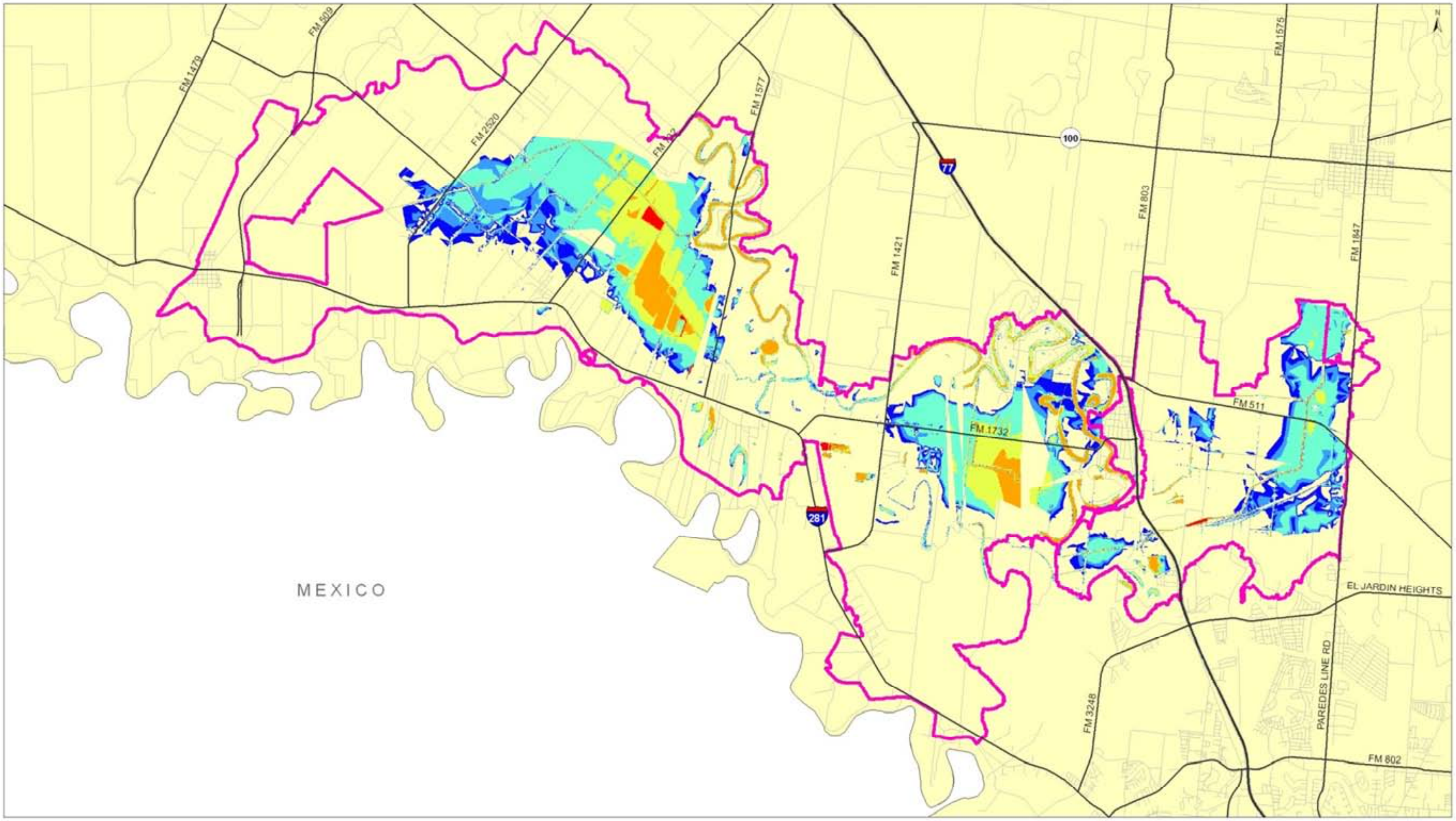
Watershed

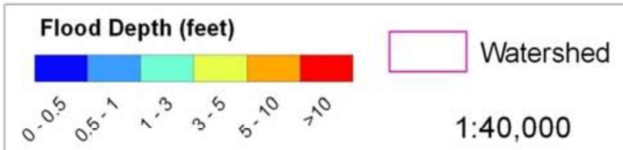
1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3 and
Upper Resaca Rancho Viejo Watersheds
Full Development - 5-yr Storm Event

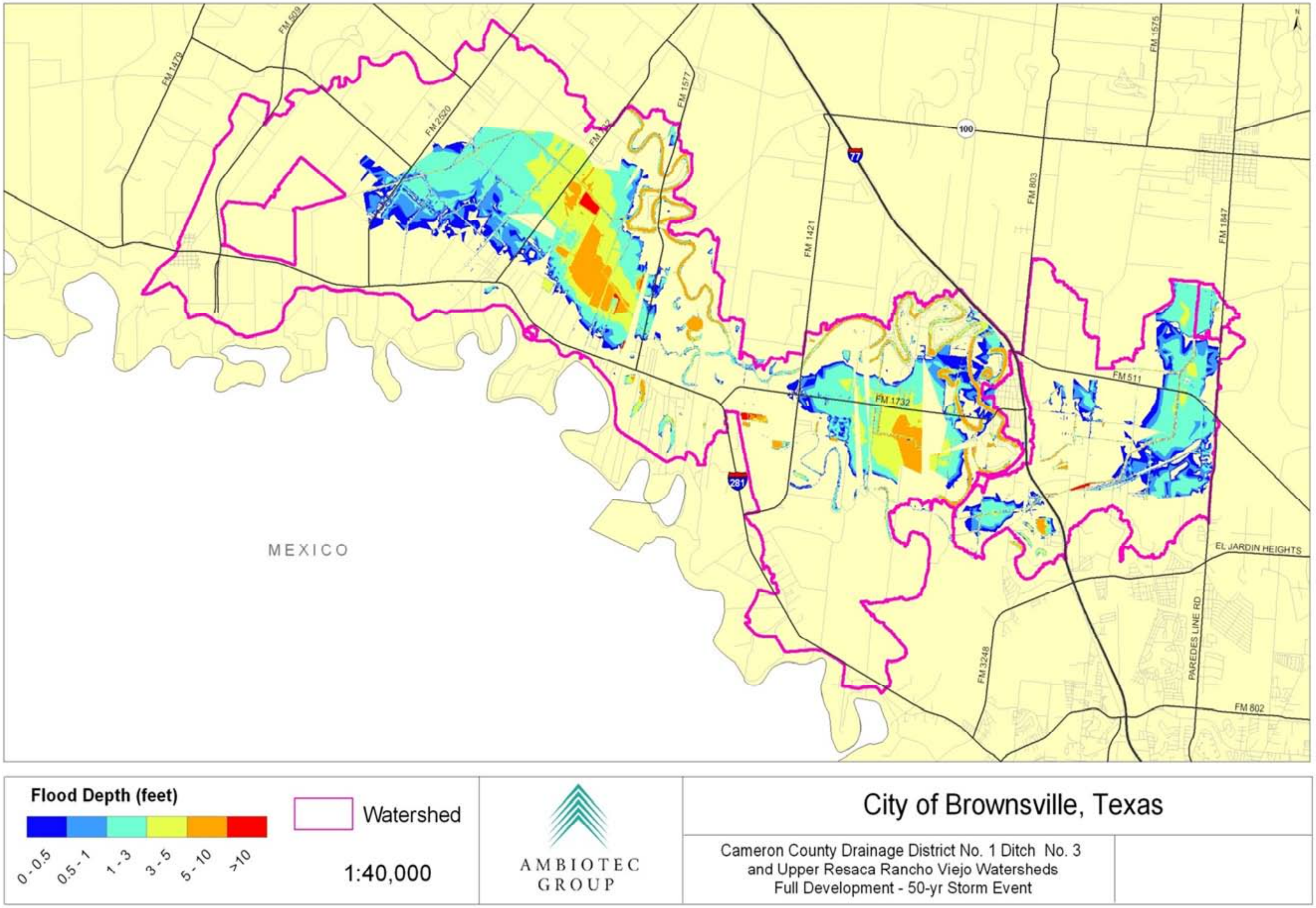




AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Full Development - 25-yr Storm Event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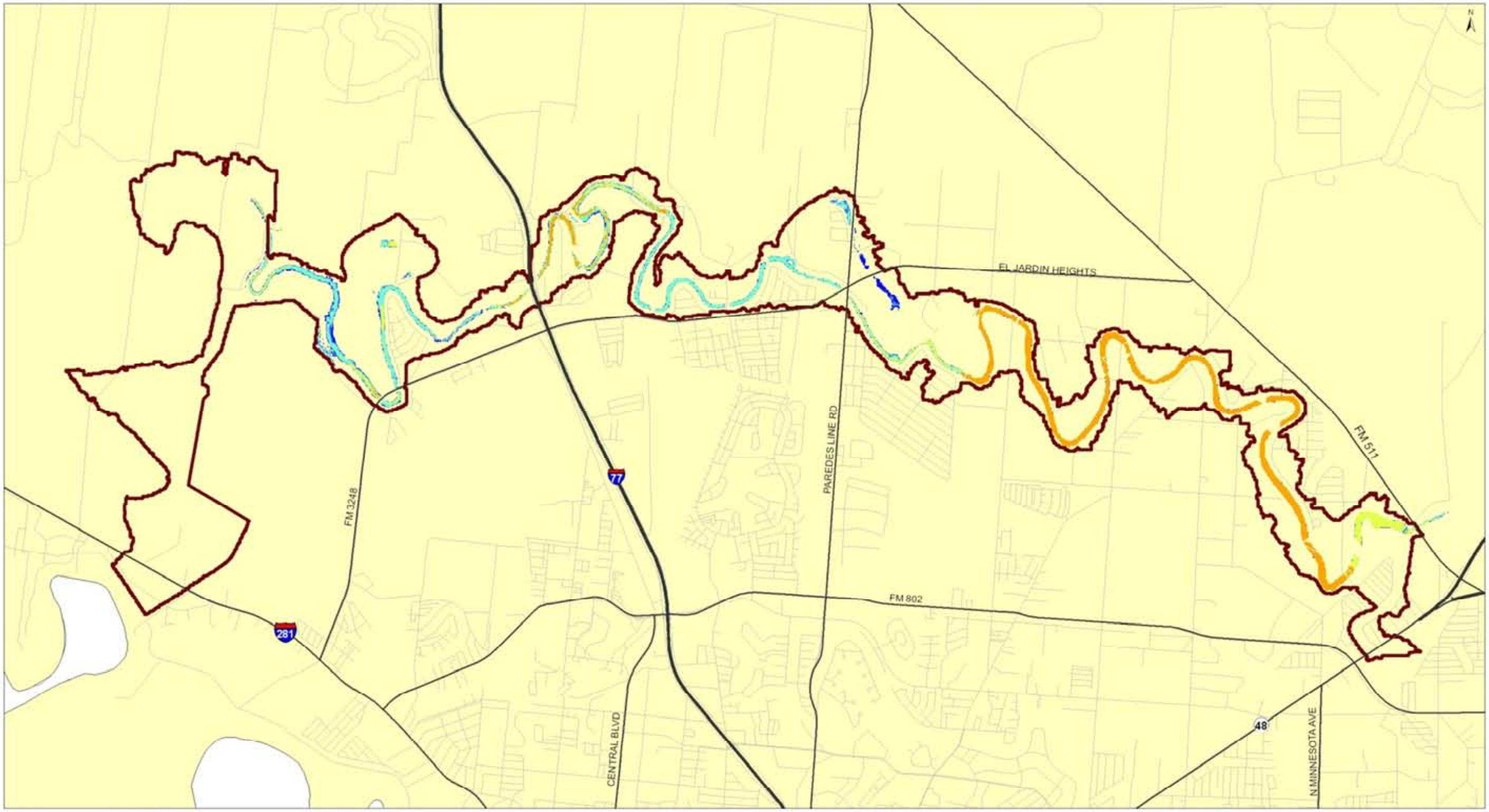
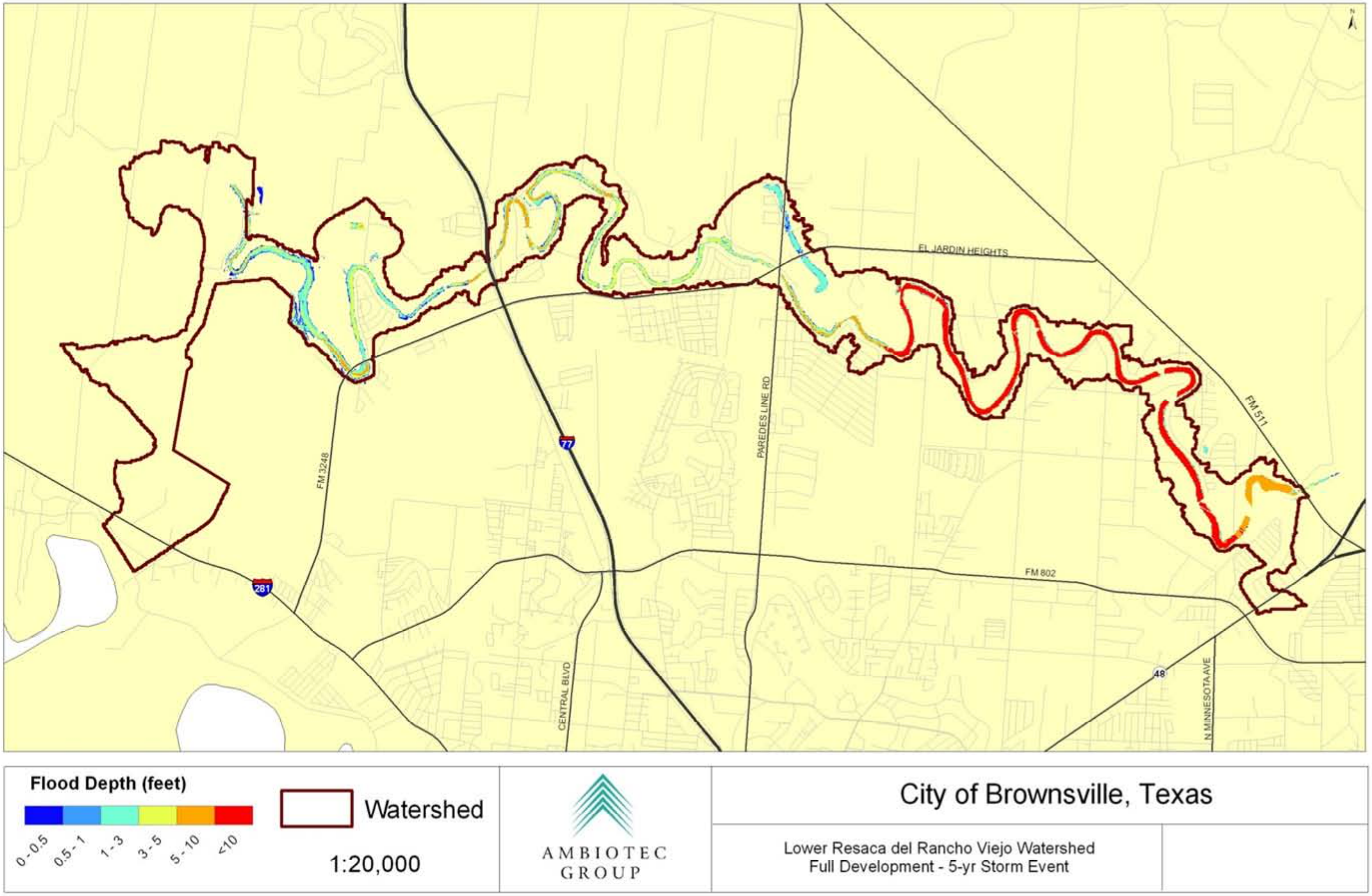
77
FM 803
FM 1575
FM 1847
FM 1421
FM 1732
FM 511
281
MEXICO
EL JARDIN HEIGHTS
FM 3248
PAREDES LINE RD
FM 802
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:40,000
AMBIOTEC GROUP
City of Brownsville, Texas
Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Full Development - 50-yr Storm Event

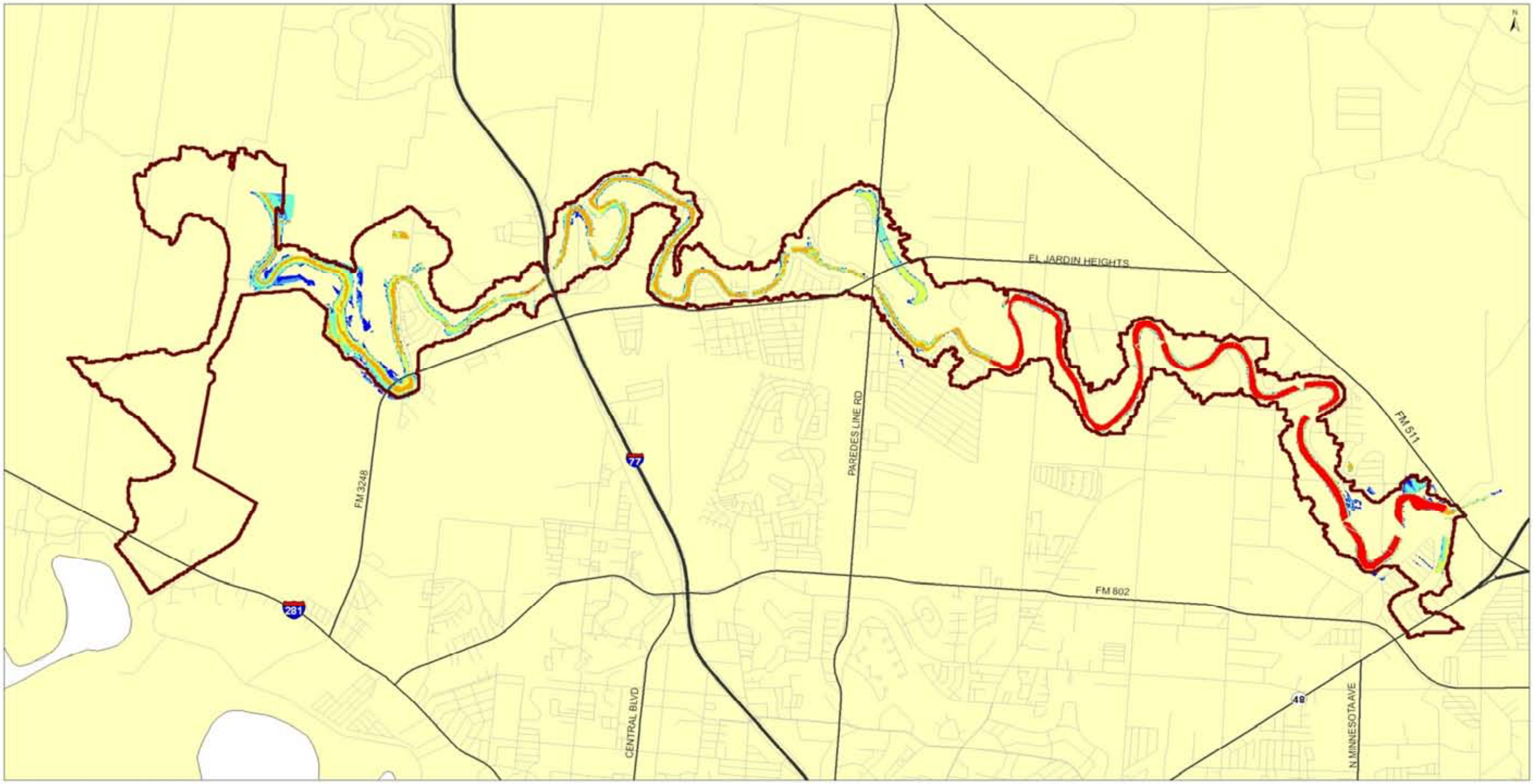


AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Full Development - 2-yr Storm Event


EL JARDIN HEIGHTS
FM 511
PAREDES LINE RD
FM 3248
77
281
FM 802
CENTRAL BLVD
48
N MINNESOTA AVE
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
<10
Watershed
1:20,000
AMBIOTEC GROUP
City of Brownsville, Texas
Lower Resaca del Rancho Viejo Watershed
Full Development - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | <10

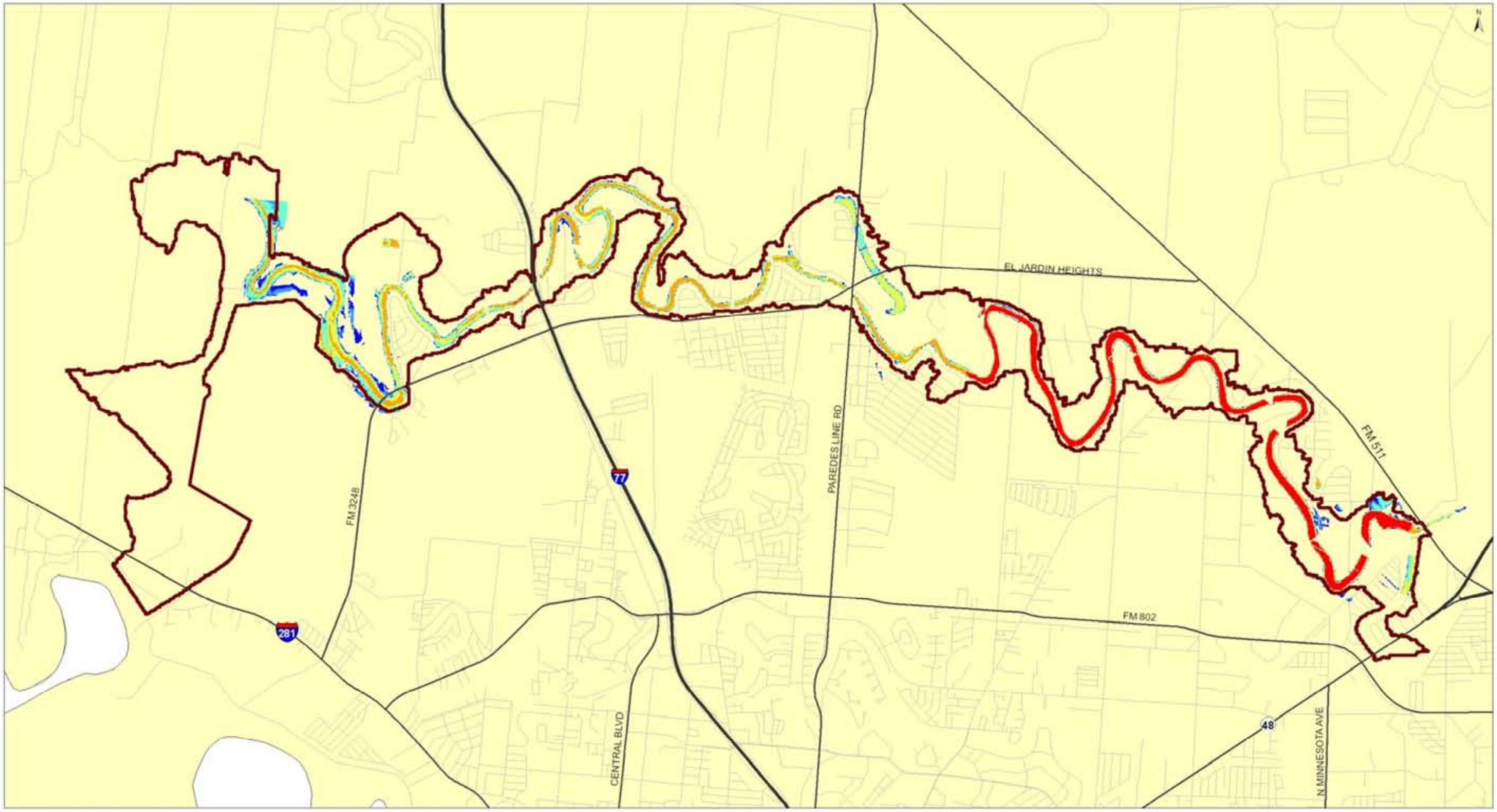
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Full Development - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | <10

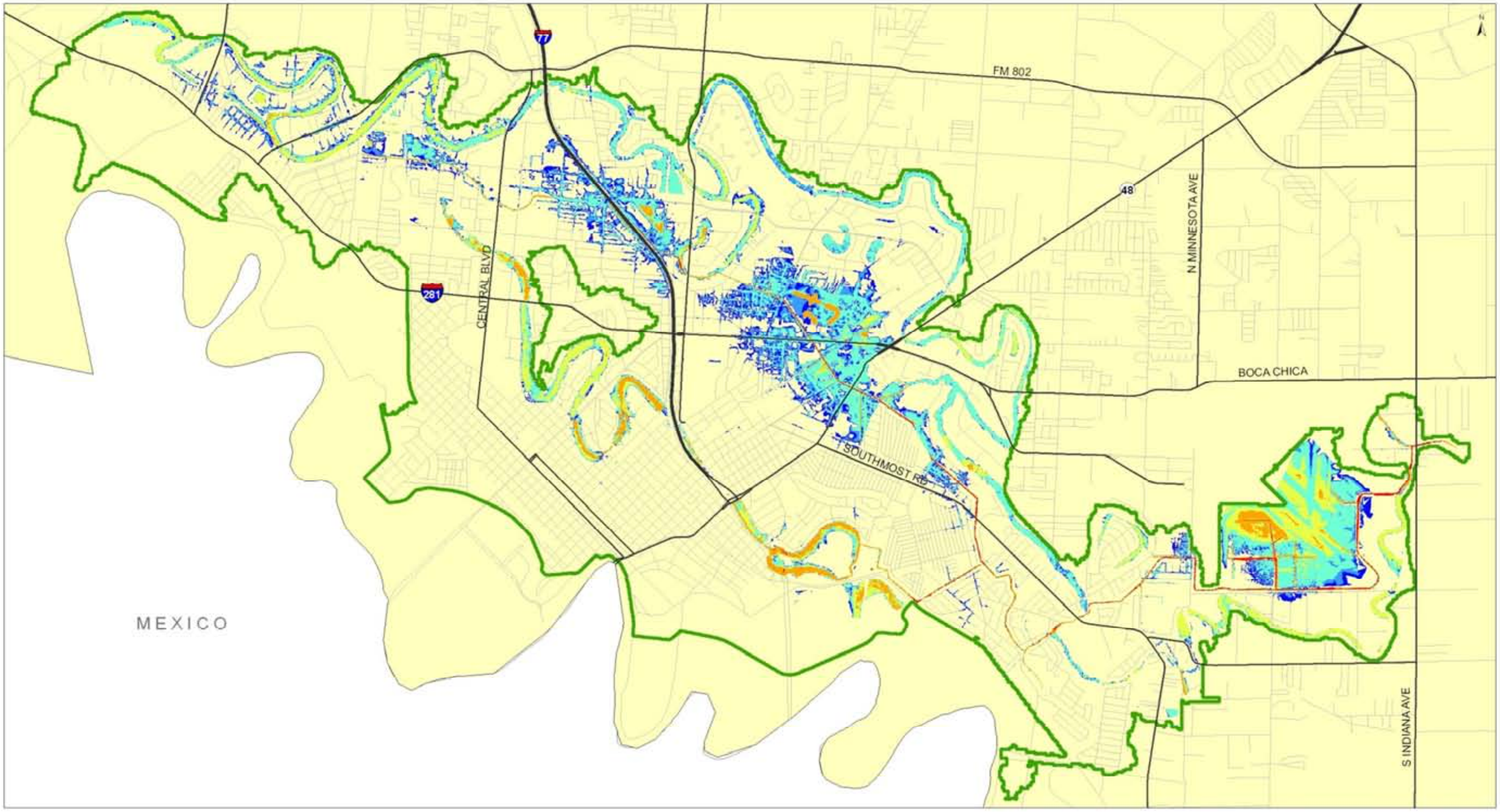
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Full Development - 50-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

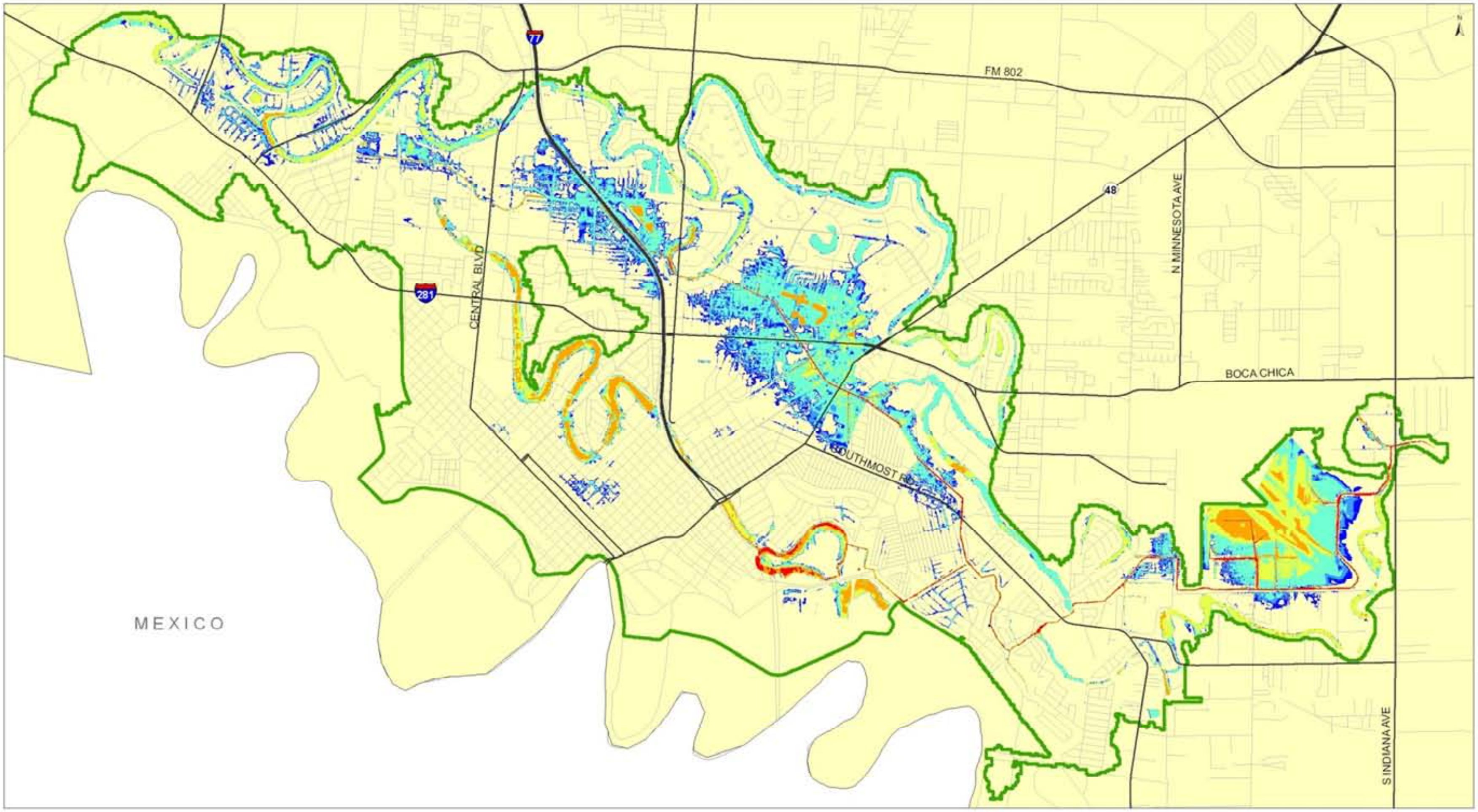
Watershed

1:20,000

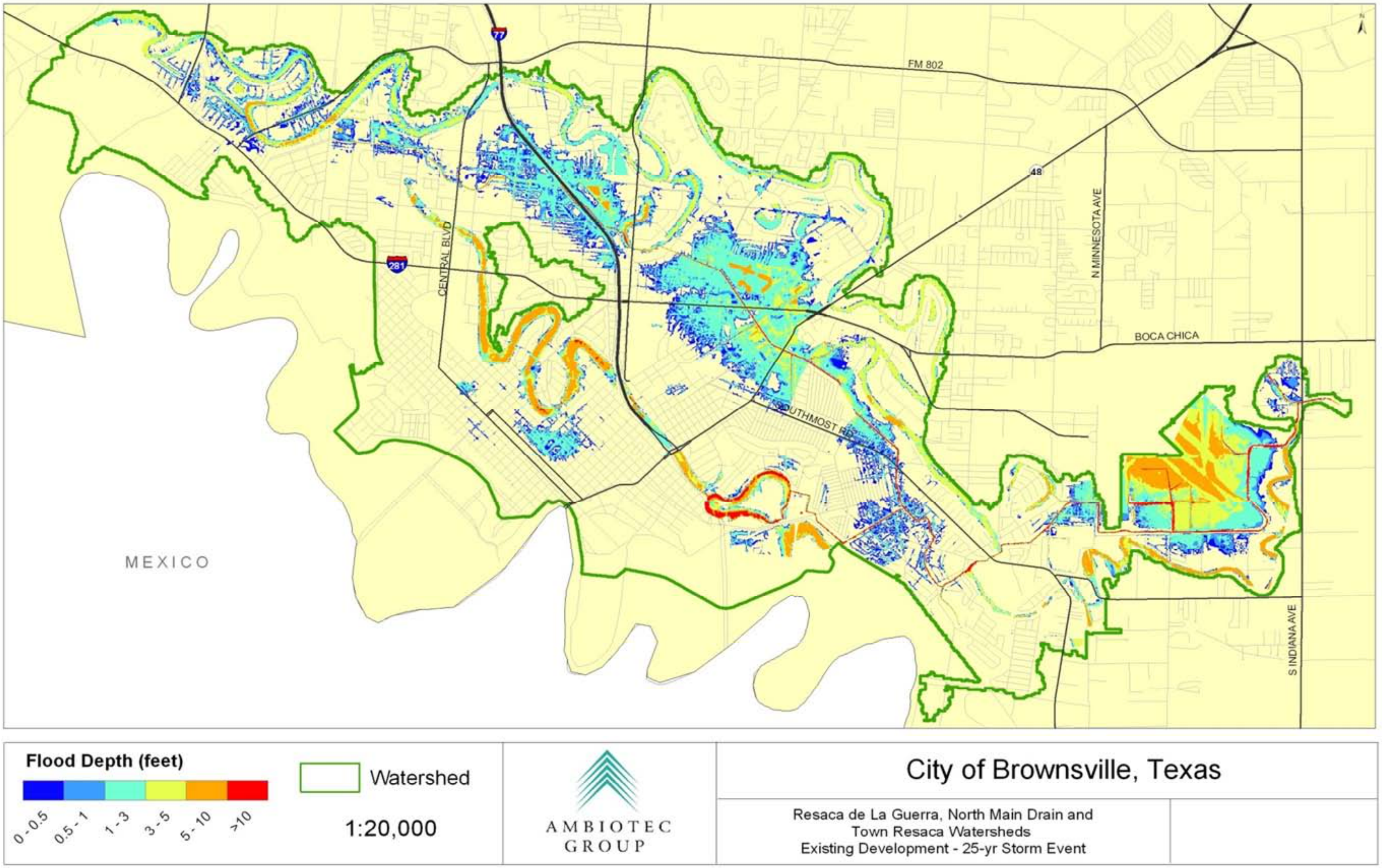
AMBIOTEC GROUP

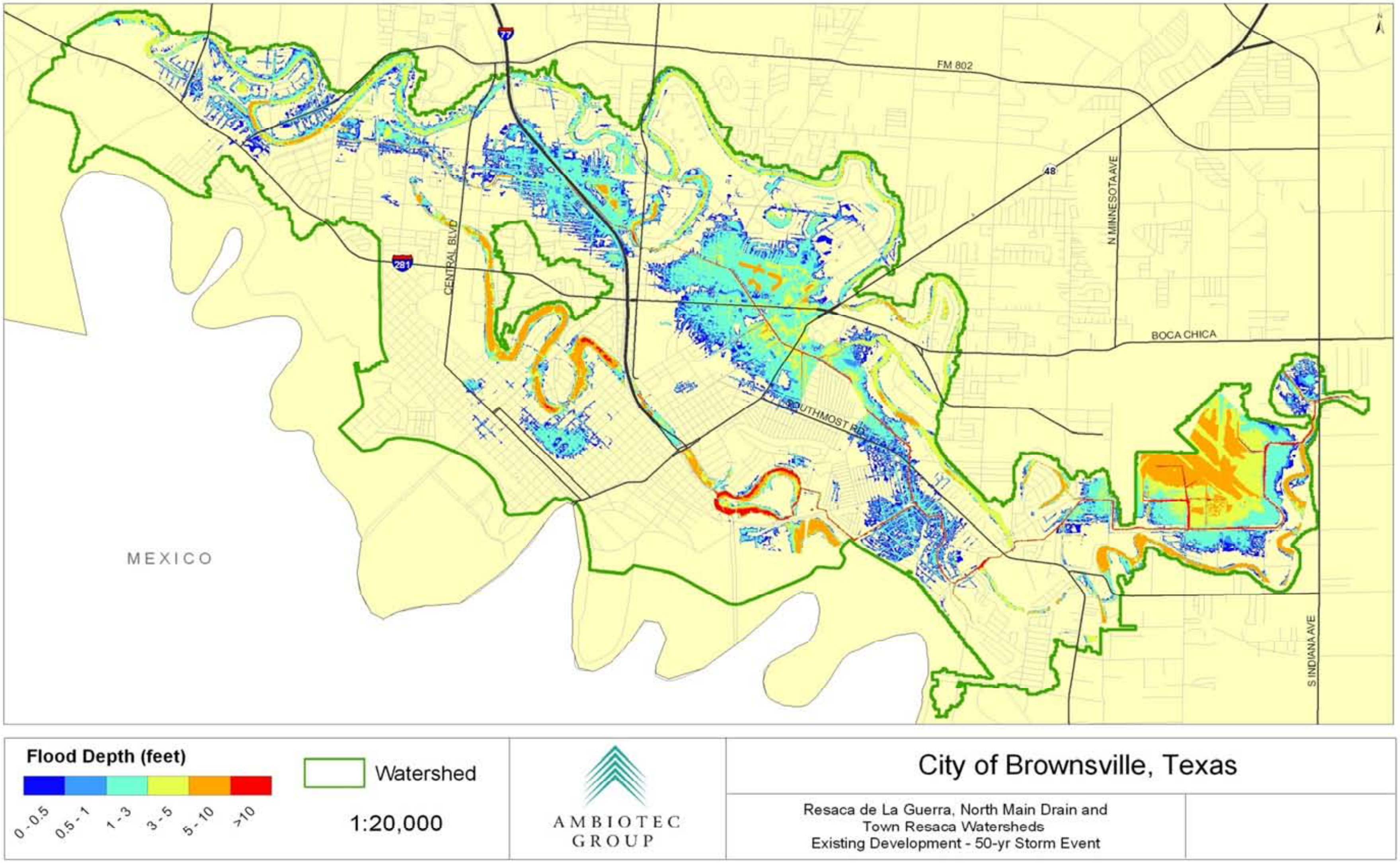
City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development - 2-yr Storm Event



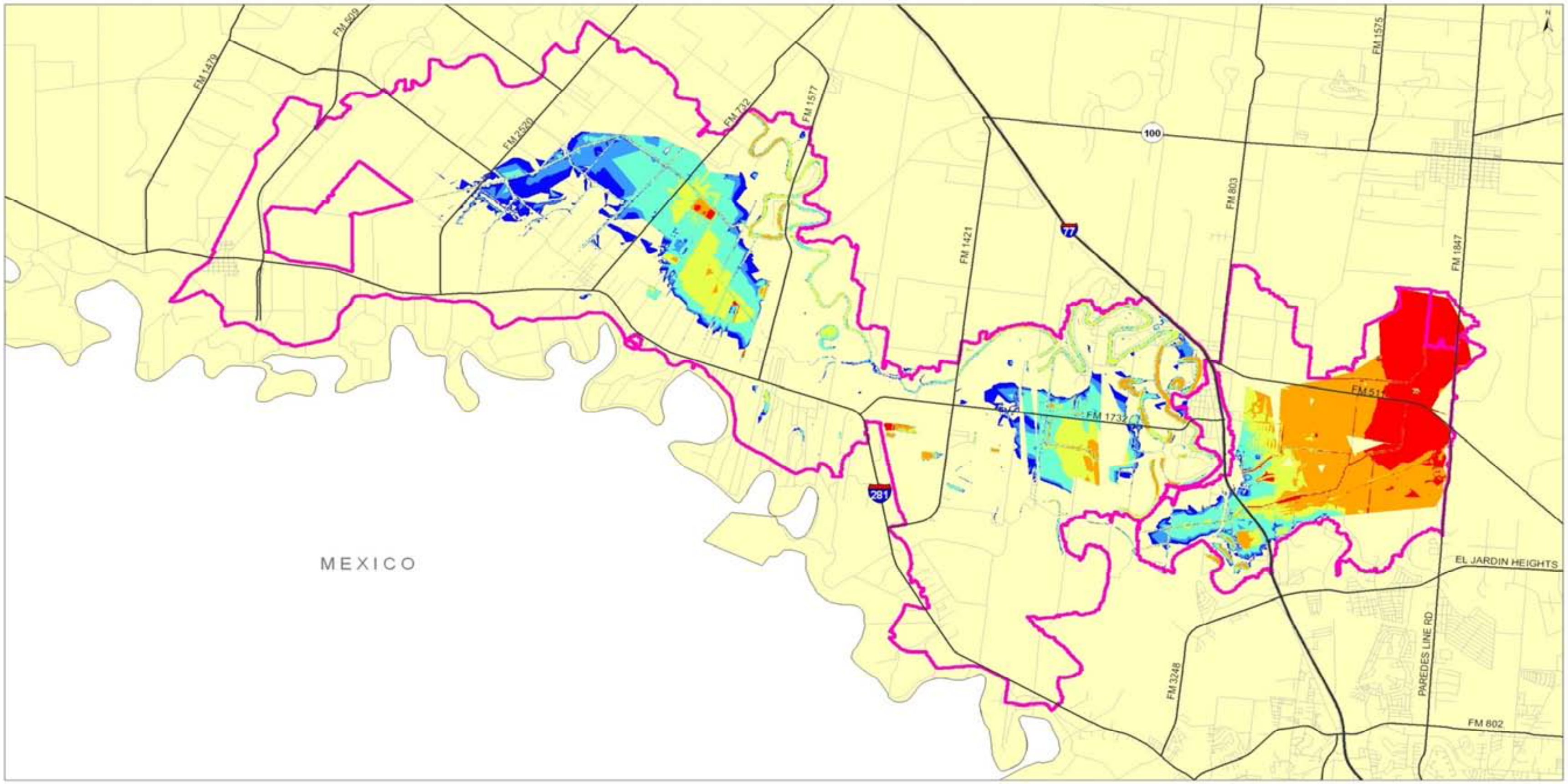
Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10	1:20,000		Resaca de La Guerra, North Main Drain and Town Resaca Watersheds Existing Development - 5-yr Storm Event


FM 802
48
N MINNESOTA AVE
CENTRAL BLVD
281
77
BOCA CHICA
SOUTHMOST RD
MEXICO
S INDIANA AVE
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:20,000
AMBIOTEC
GROUP
City of Brownsville, Texas
Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development - 25-yr Storm Event



FM 802
48
N MINNESOTA AVE
BOCA CHICA
281
CENTRAL BLVD
77
SOUTHMOST RD
S INDIANA AVE
MEXICO
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:20,000
AMBIOTEC GROUP
City of Brownsville, Texas
Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development - 50-yr Storm Event

APPENDIX B



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

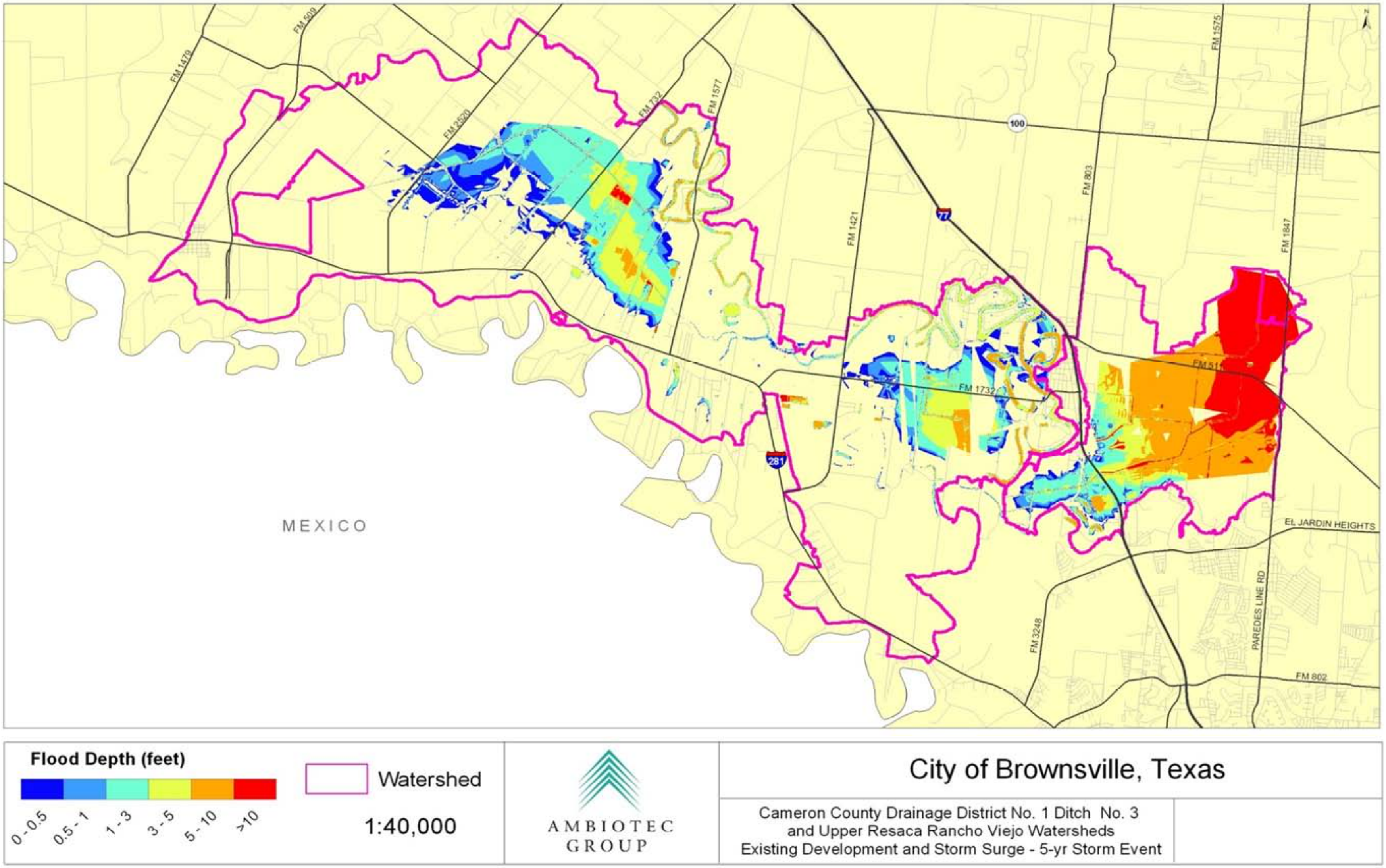
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development and Storm Surge - 2-yr Storm Event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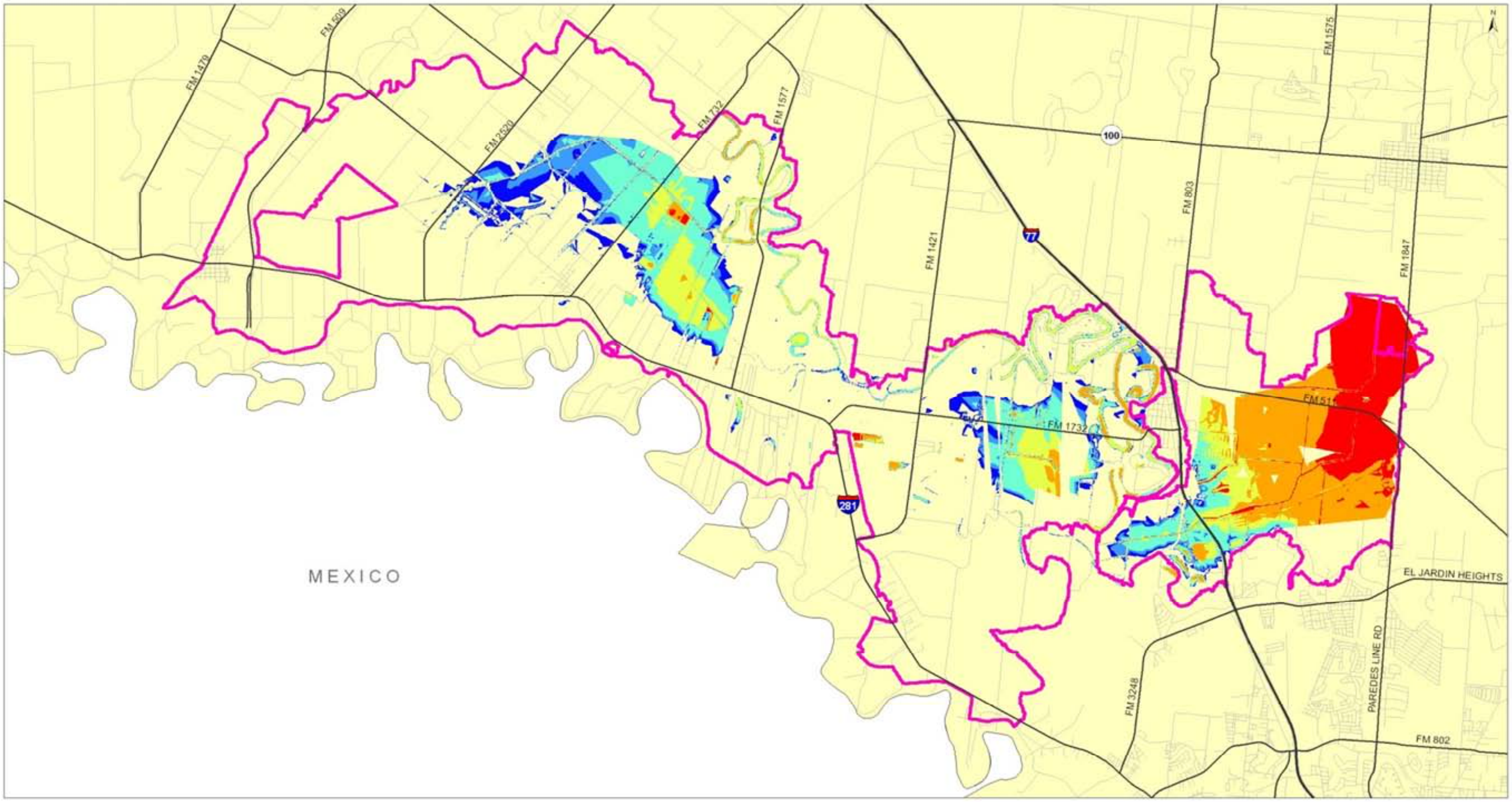
FM 1847
FM 511
FM 1732
281
MEXICO
EL JARDIN HEIGHTS
FM 3248
PAREDES LINE RD
FM 802
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:40,000
AMBIOTEC GROUP
City of Brownsville, Texas
Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development and Storm Surge - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

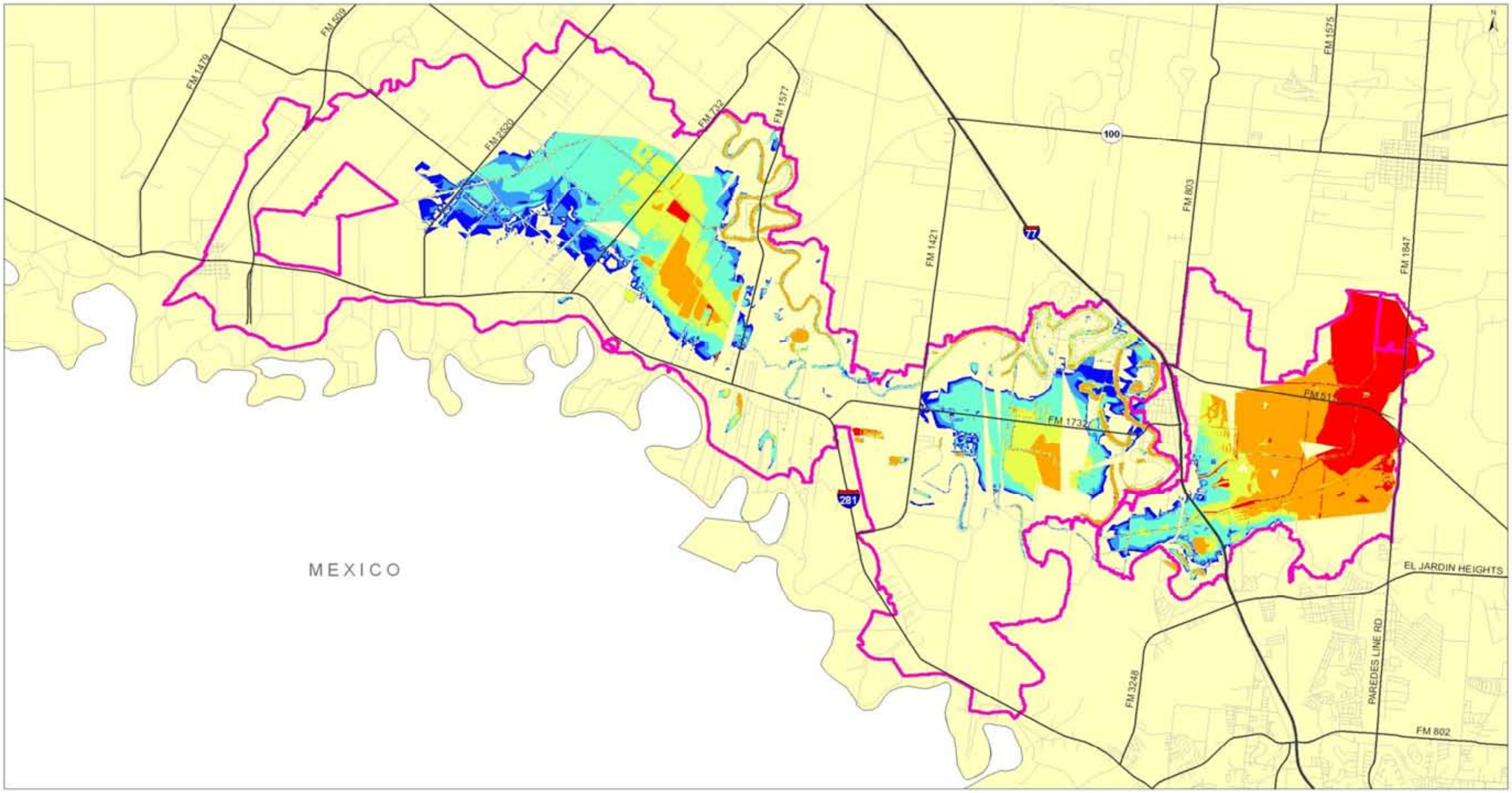
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3 and
Upper Resaca Rancho Viejo Watersheds
Existing Development and Storm Surge - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

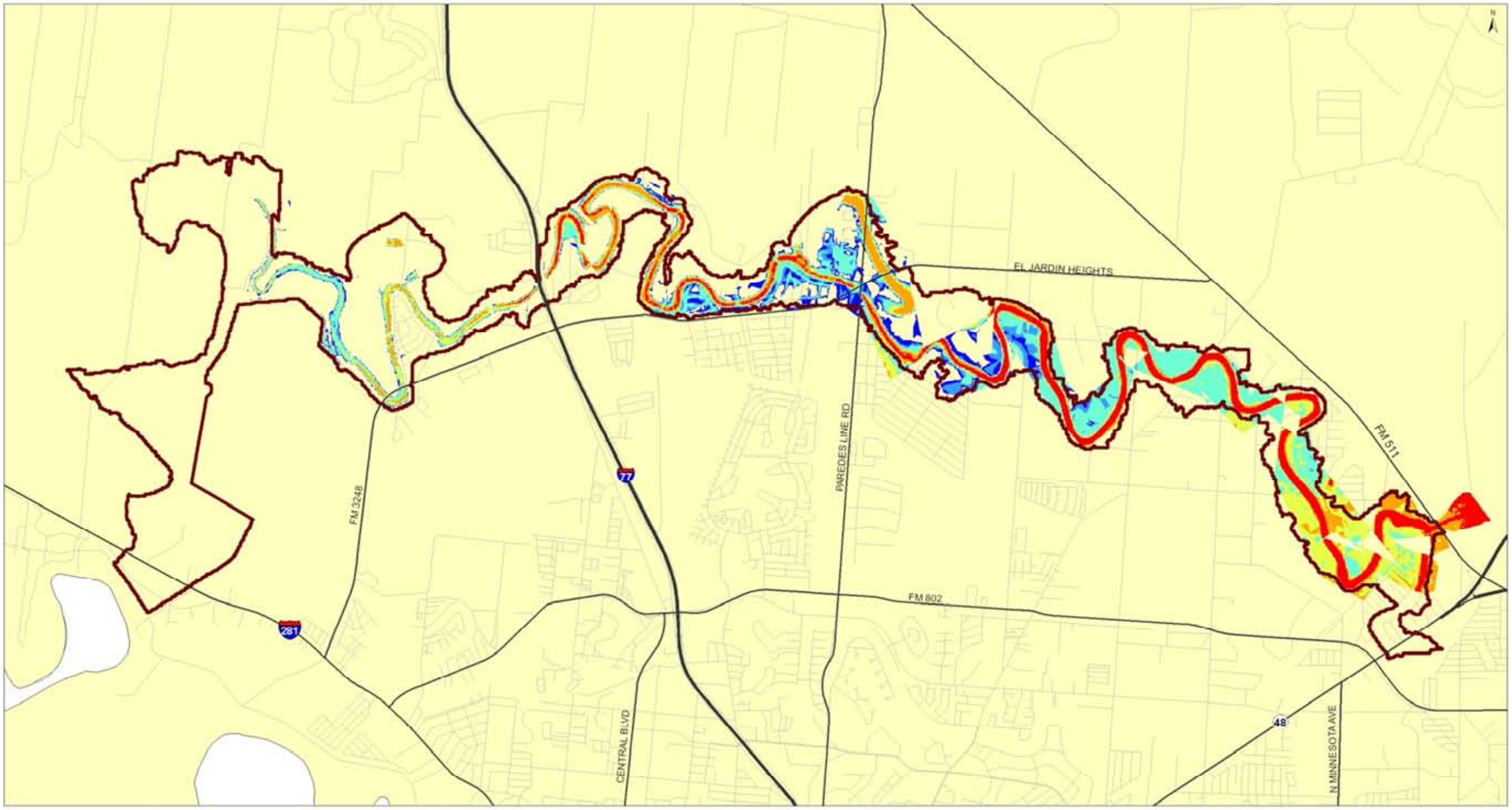
Watershed

1:40,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District No. 1 Ditch No. 3
and Upper Resaca Rancho Viejo Watersheds
Existing Development and Storm Surge - 50-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

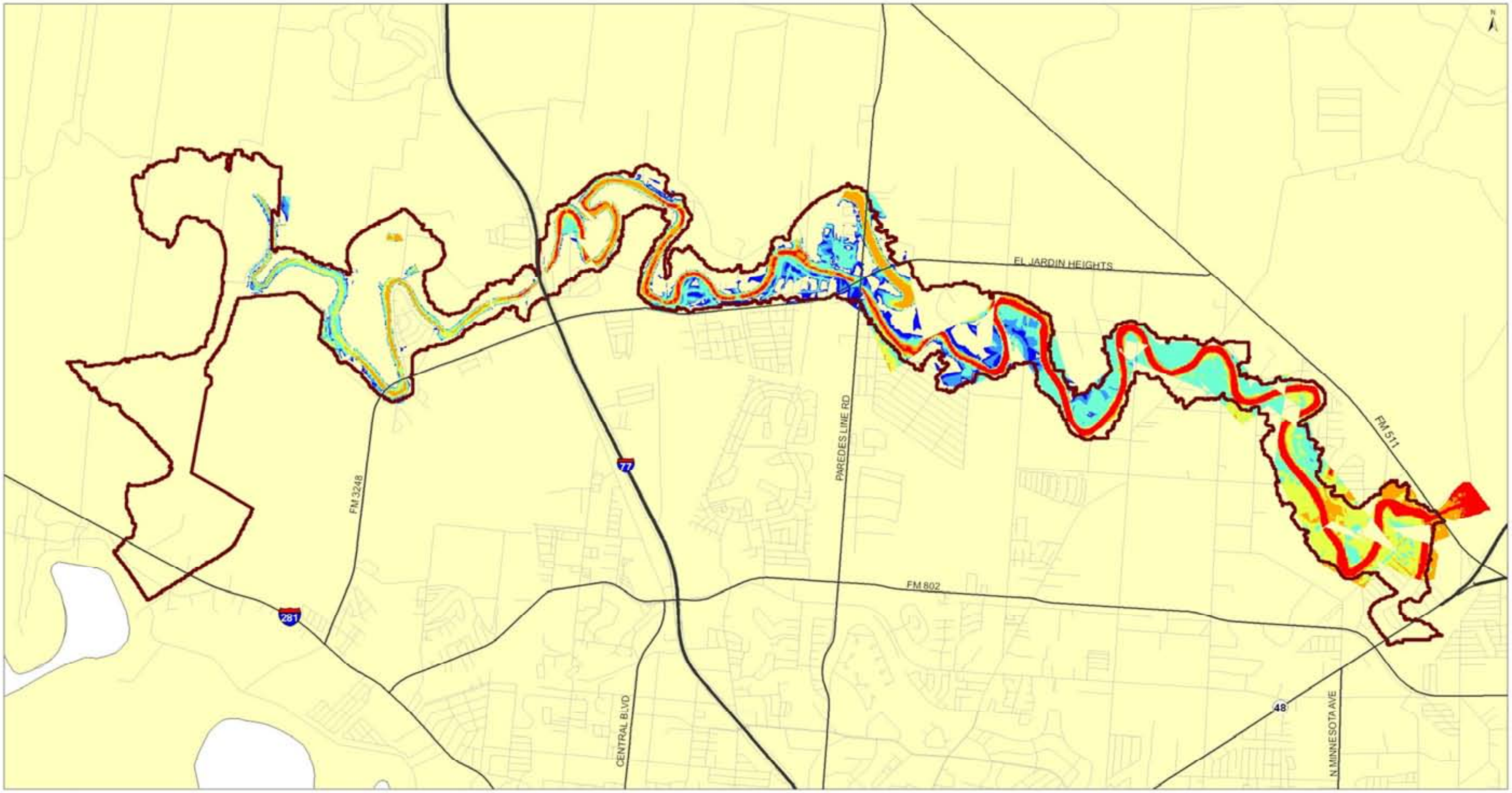
Watershed

1:20,000

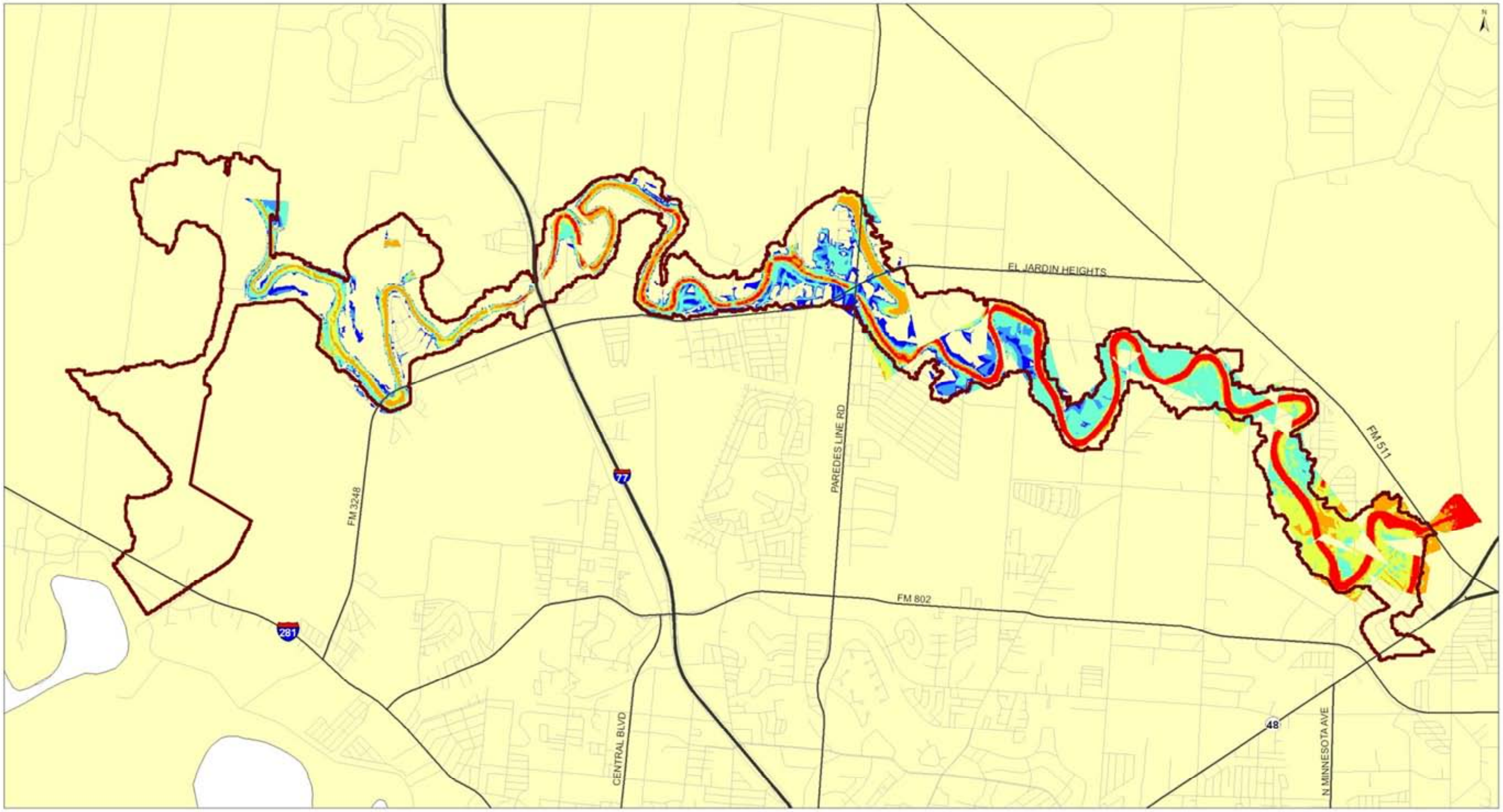
AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Existing Development and Storm Surge - 2-yr Storm Event



Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10	1:20,000		Lower Resaca del Rancho Viejo Watershed Existing Development and Storm Surge - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

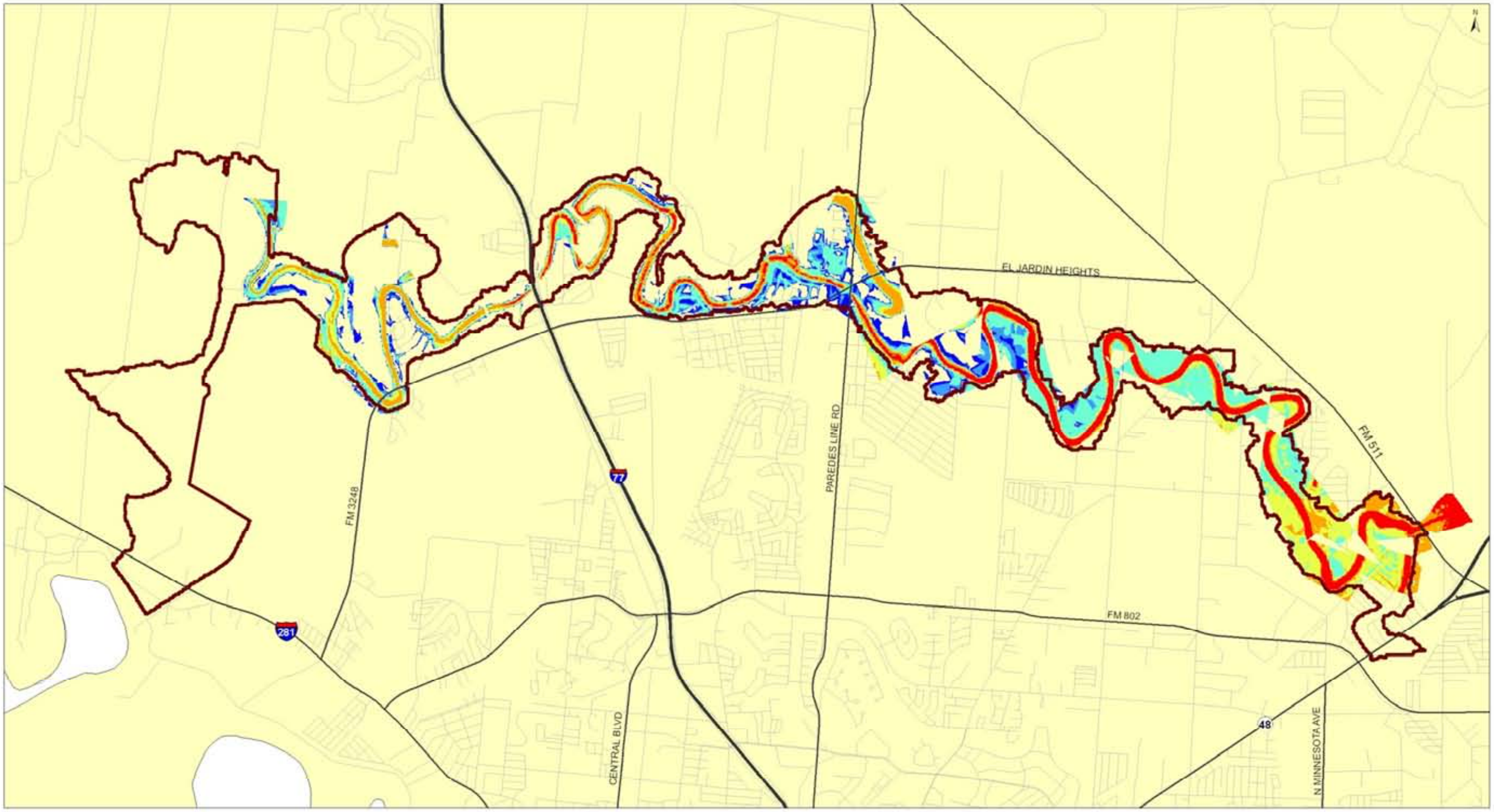
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Existing Development and Storm Surge - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

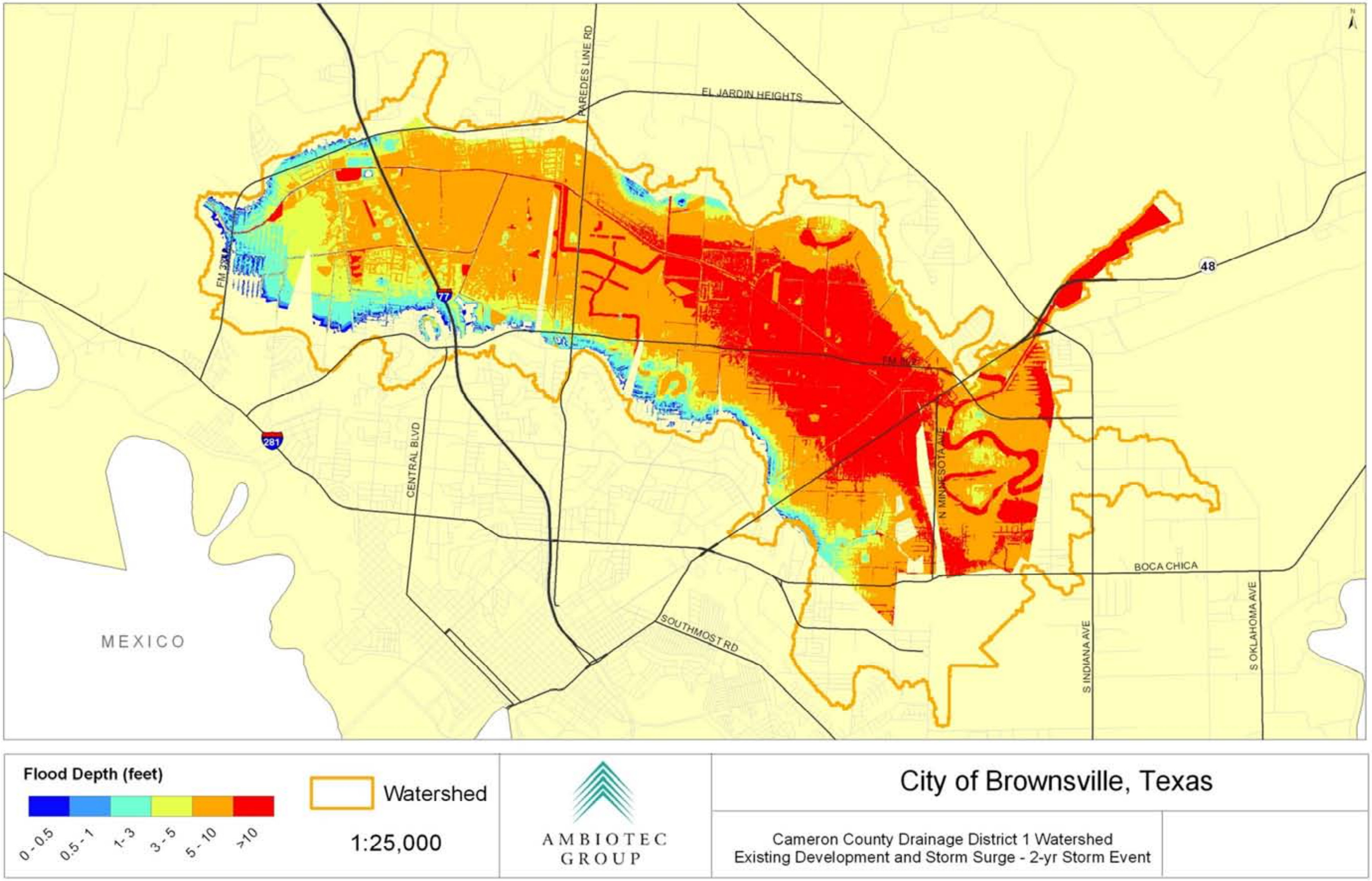
Watershed

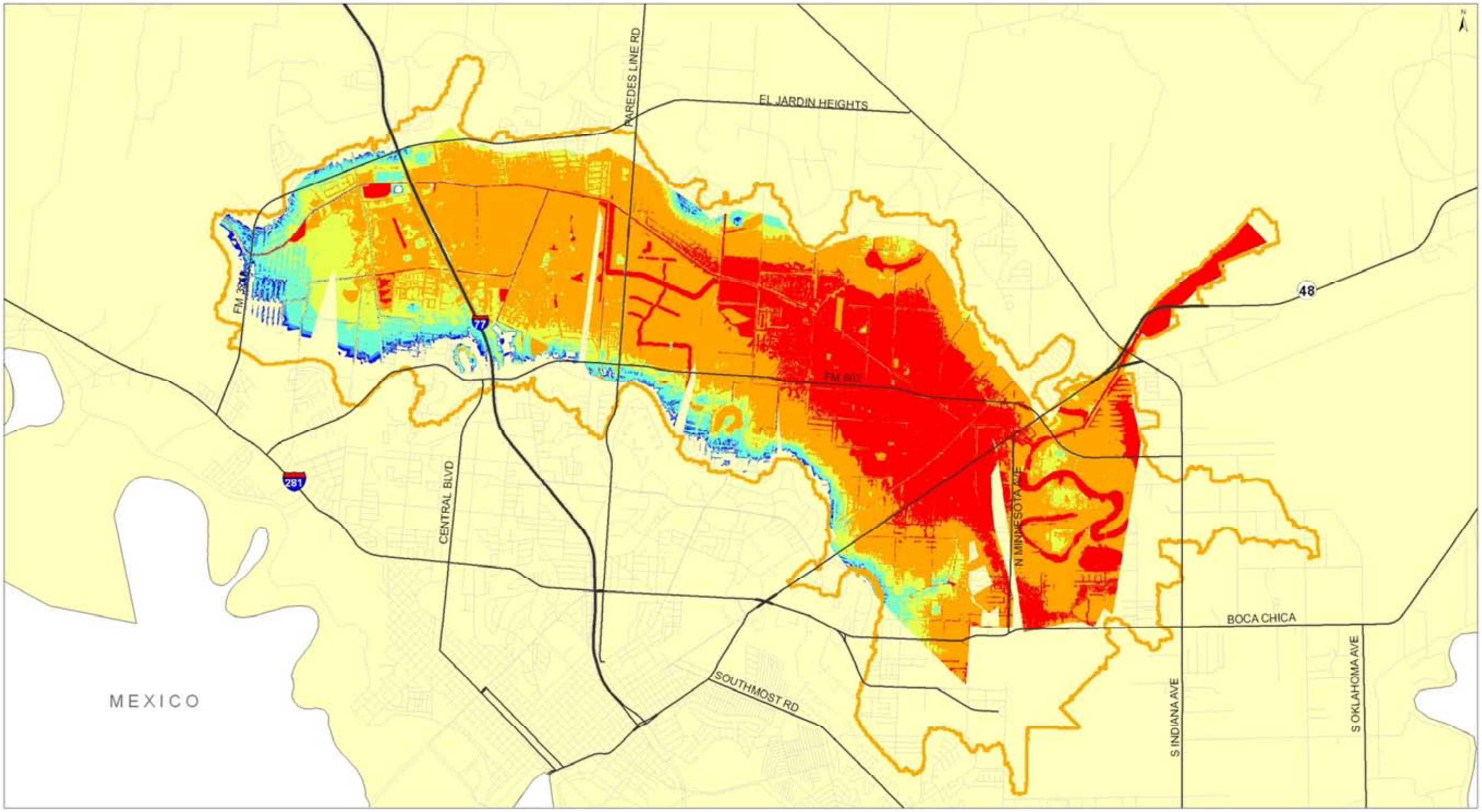
1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Lower Resaca del Rancho Viejo Watershed
Existing Development and Storm Surge - 50-yr Storm Event


EL JARDIN HEIGHTS
PAREDES LINE RD
77
281
48
CENTRAL BLVD
N MINNESOTA AVE
BOCA CHICA
SOUTHMOST RD
S INDIANA AVE
S OKLAHOMA AVE
MEXICO
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:25,000
AMBIOTEC GROUP
City of Brownsville, Texas
Cameron County Drainage District 1 Watershed
Existing Development and Storm Surge - 2-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

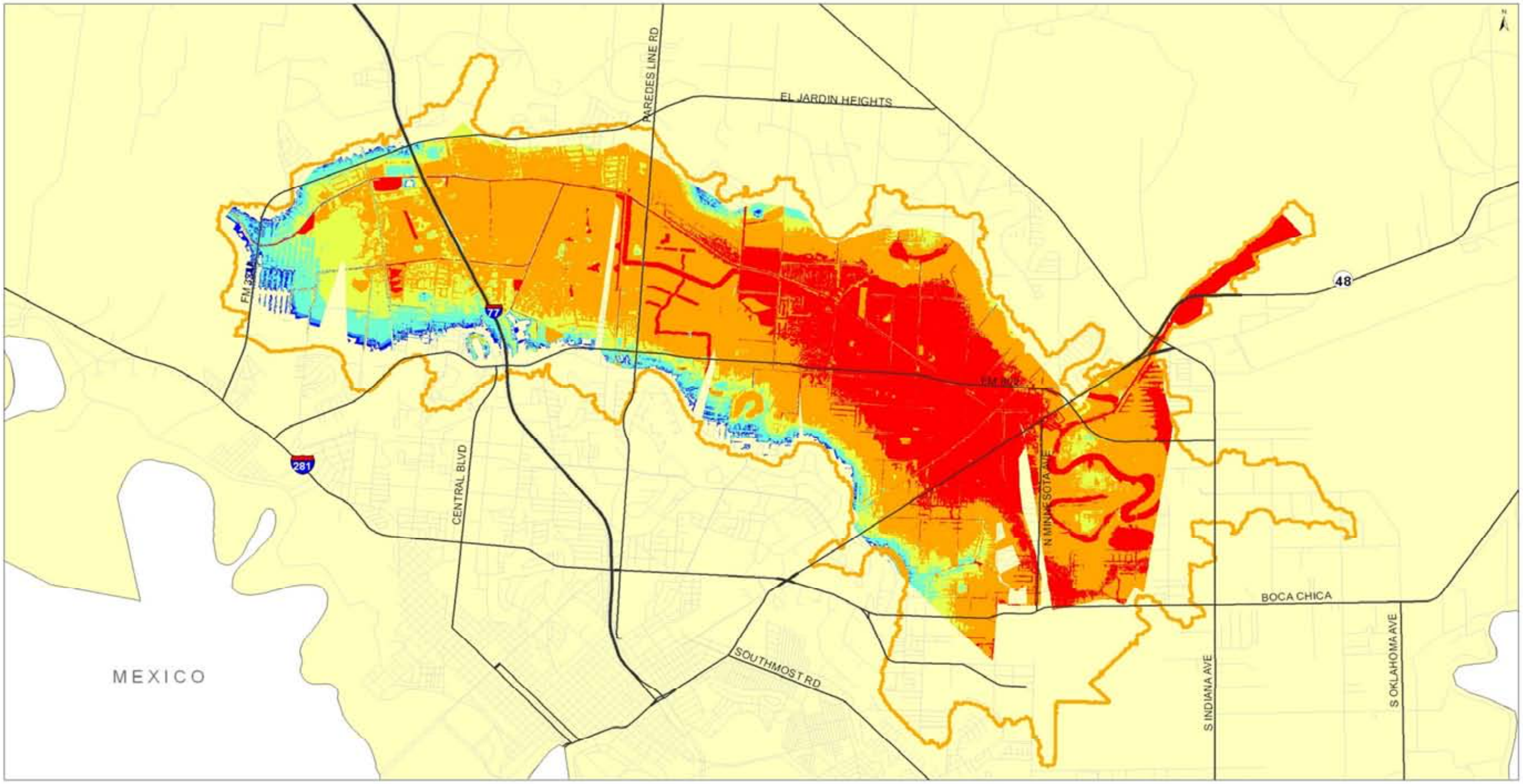
Watershed

1:25,000

AMBIOTEC GROUP

City of Brownsville, Texas

Cameron County Drainage District 1 Watershed
Existing Development and Storm Surge - 5-yr Storm Event



Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
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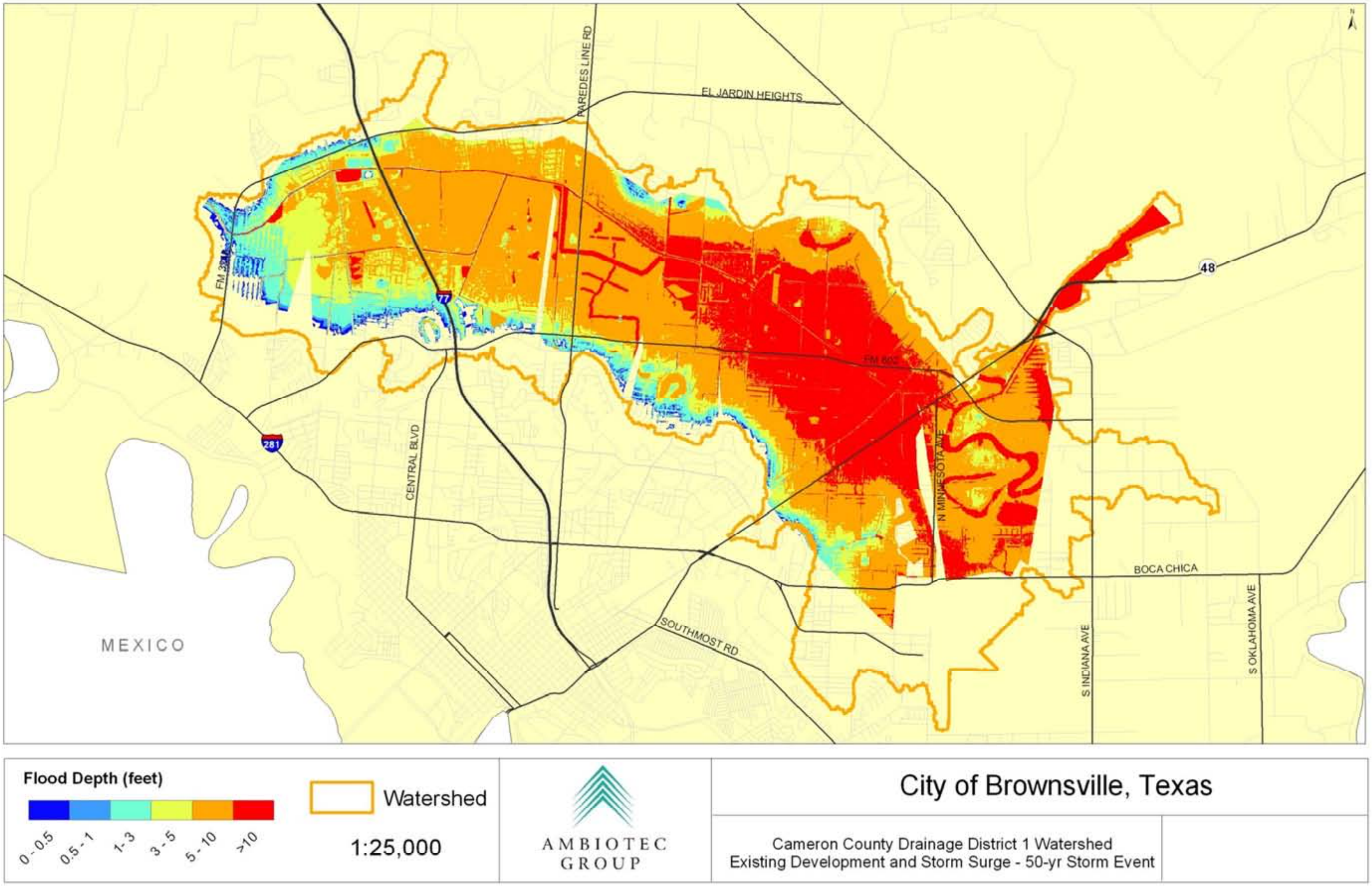
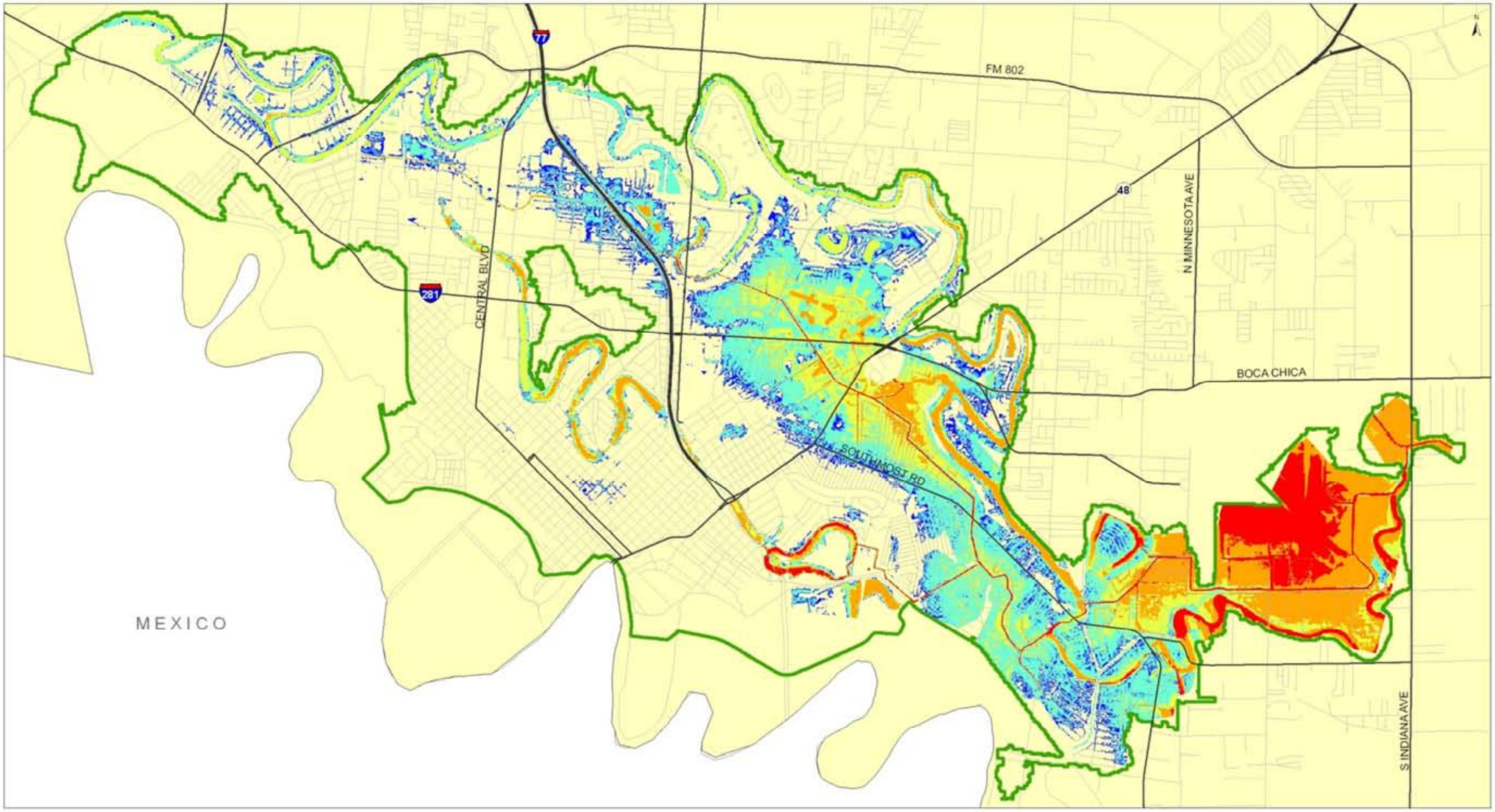


EL JARDIN HEIGHTS
PAREDES LINE RD
FM 3248
77
48
FM 802
N MINNESOTA AVE
281
CENTRAL BLVD
BOCA CHICA
S OKLAHOMA AVE
S INDIANA AVE
SOUTHMOST RD
MEXICO
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:25,000
AMBIOTEC GROUP
City of Brownsville, Texas
Cameron County Drainage District 1 Watershed
Existing Development and Storm Surge - 50-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

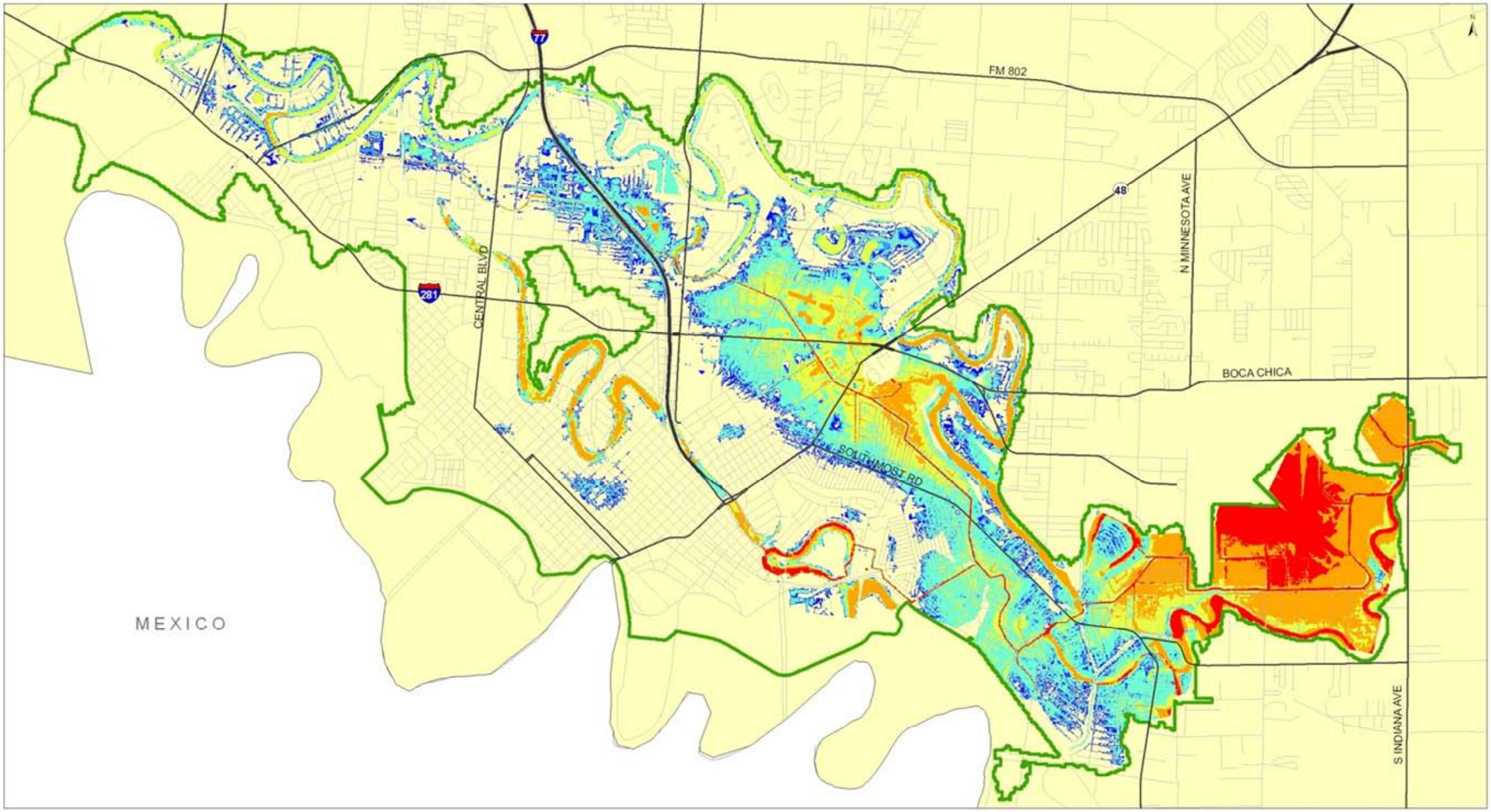
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Storm Surge - 2-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

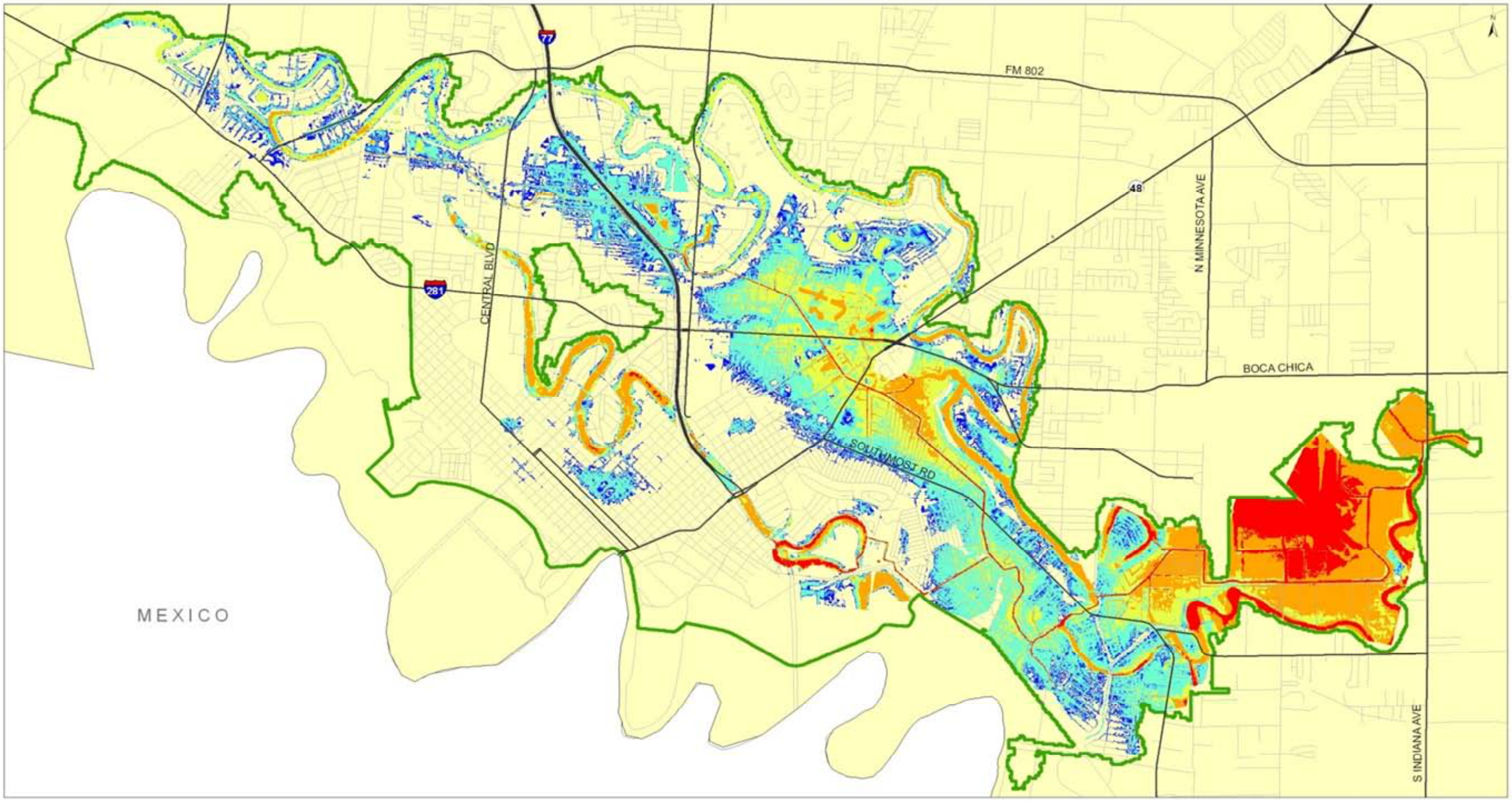
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Storm Surge - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

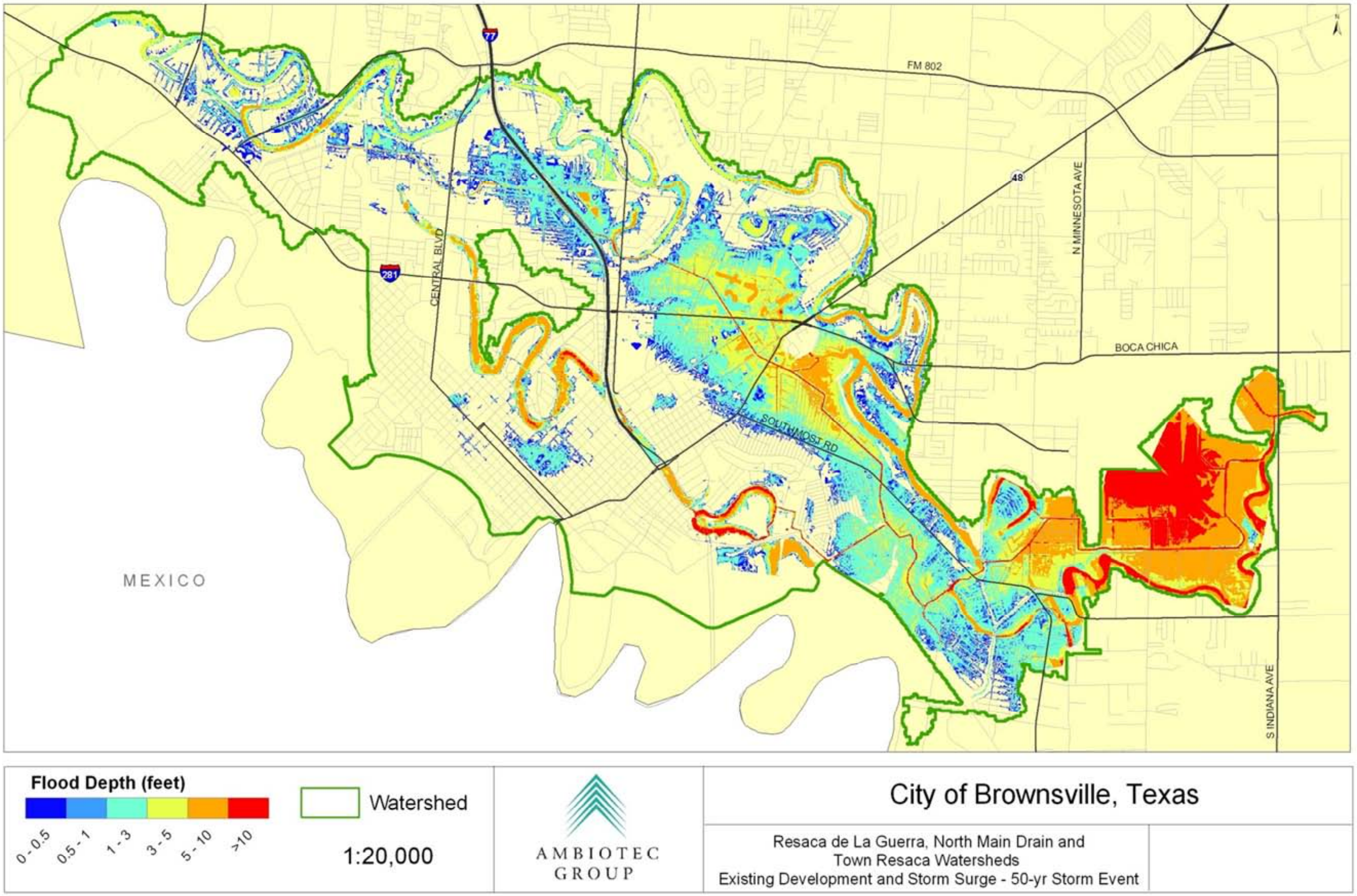
Watershed

1:20,000

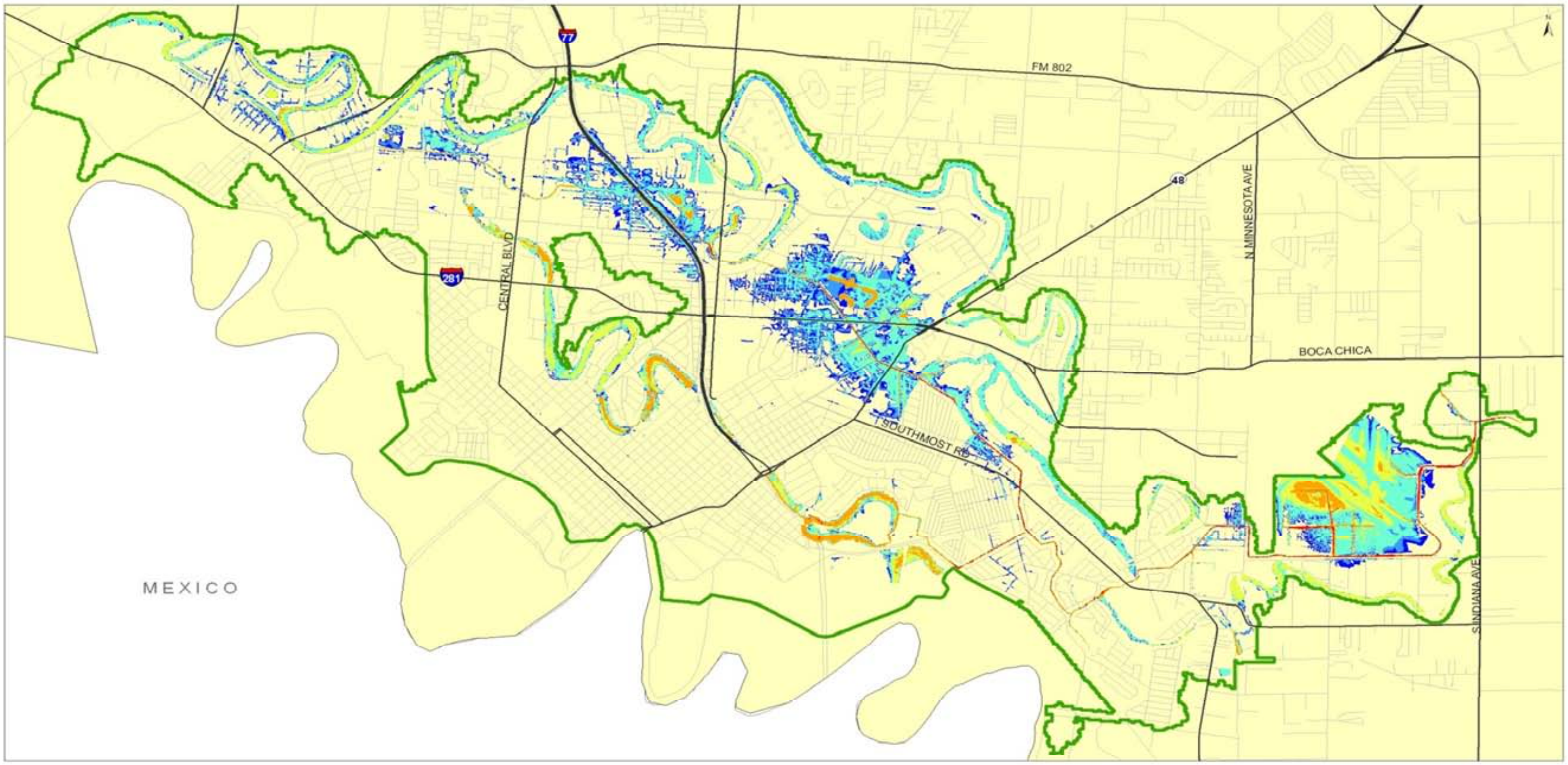
AMBIOTEC GROUP

City of Brownsville, Texas

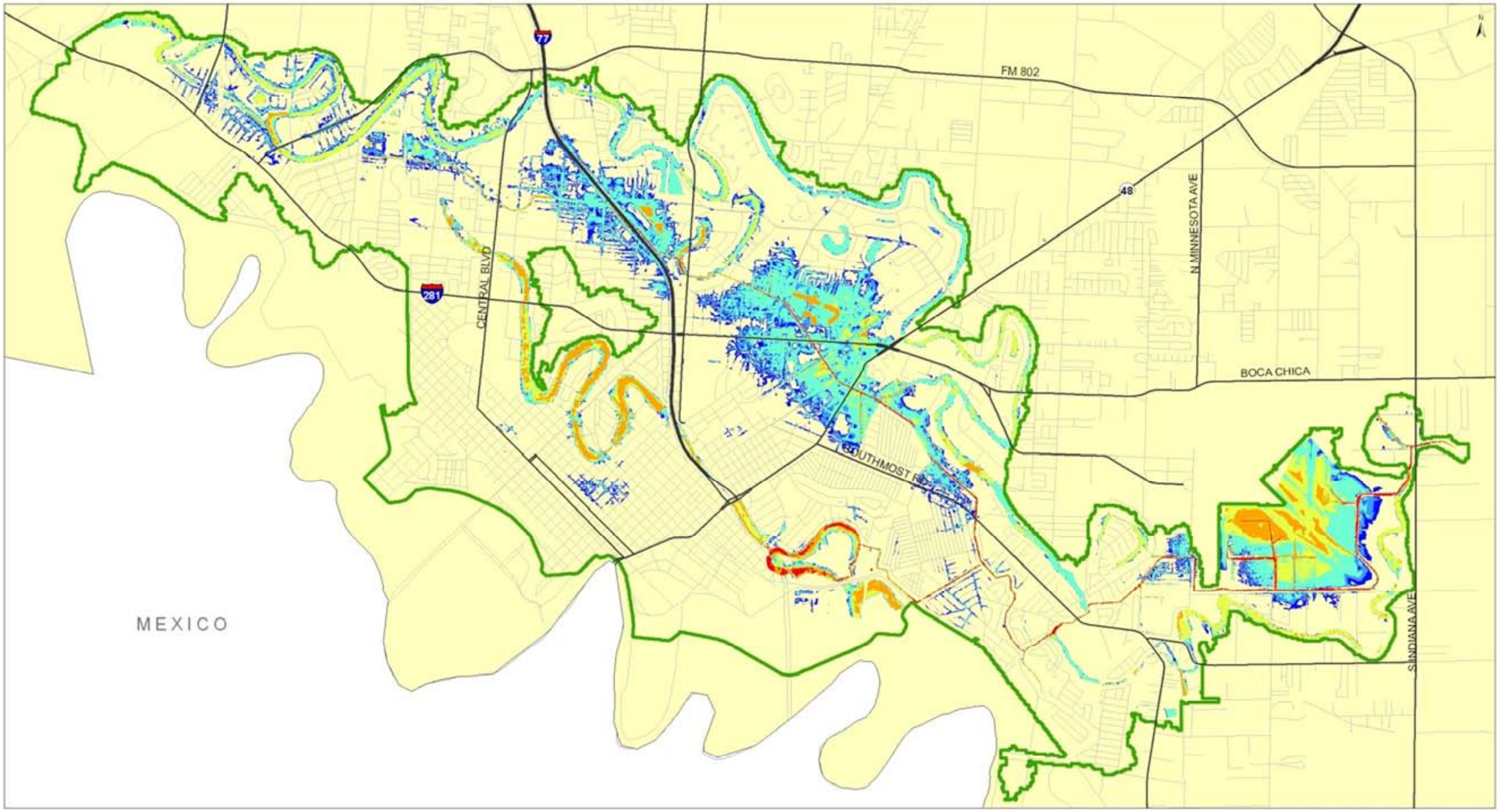
Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Storm Surge - 25-yr Storm Event


FM 802
48
N MINNESOTA AVE
BOCA CHICA
CENTRAL BLVD
281
77
SOUTHMOST RD
S INDIANA AVE
MEXICO
Flood Depth (feet)
0 - 0.5
0.5 - 1
1 - 3
3 - 5
5 - 10
>10
Watershed
1:20,000
AMBIOTEC
GROUP
City of Brownsville, Texas
Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Storm Surge - 50-yr Storm Event

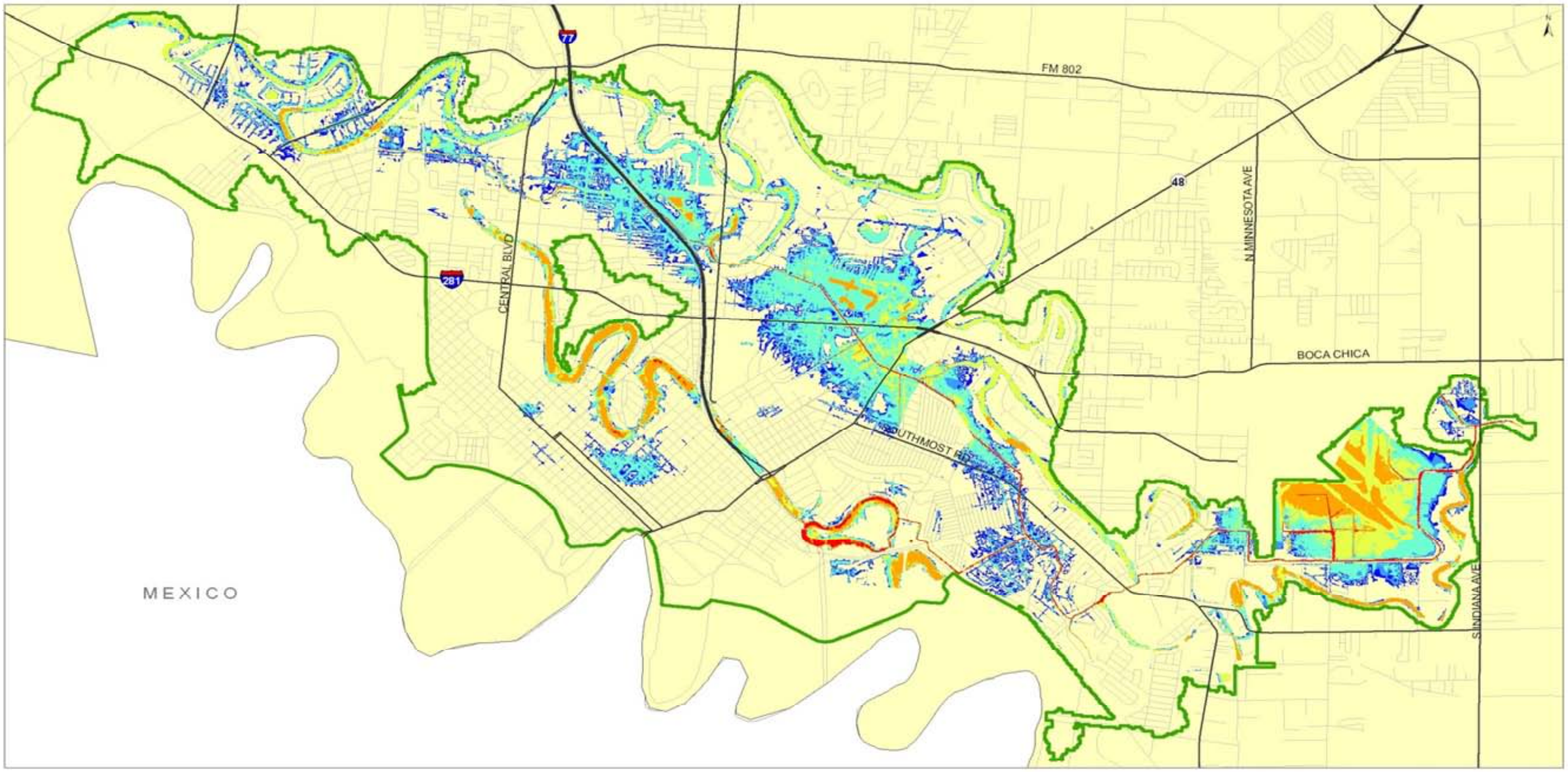
APPENDIX C



Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10	1:20,000		Resaca de La Guerra, North Main Drain and Town Resaca Watersheds Existing Development and Culvert Upgrades - 2-yr Storm Event



Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10	1:20,000		Resaca de La Guerra, North Main Drain and Town Resaca Watersheds Existing Development and Culvert Upgrades - 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

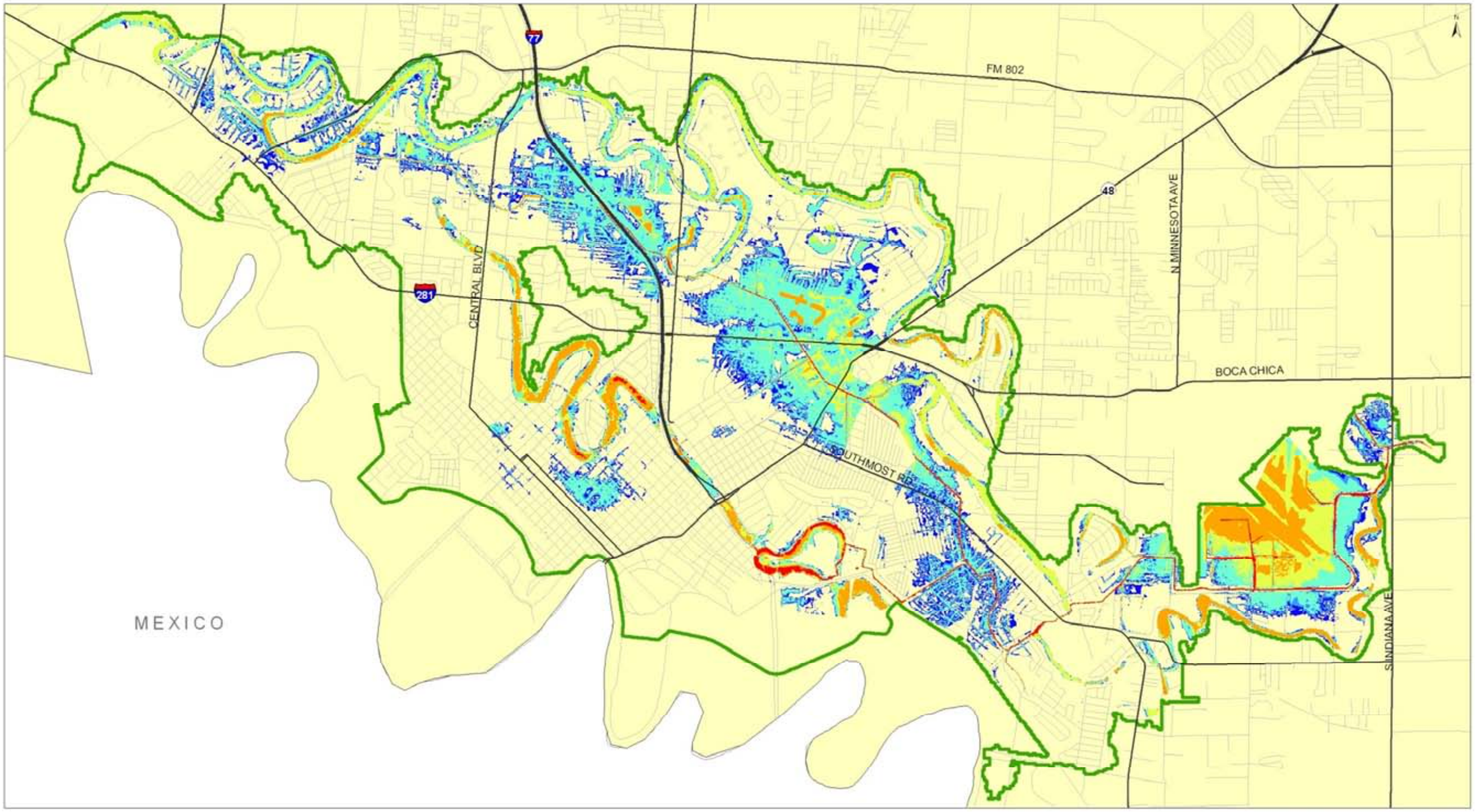
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Culvert Upgrades - 25-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

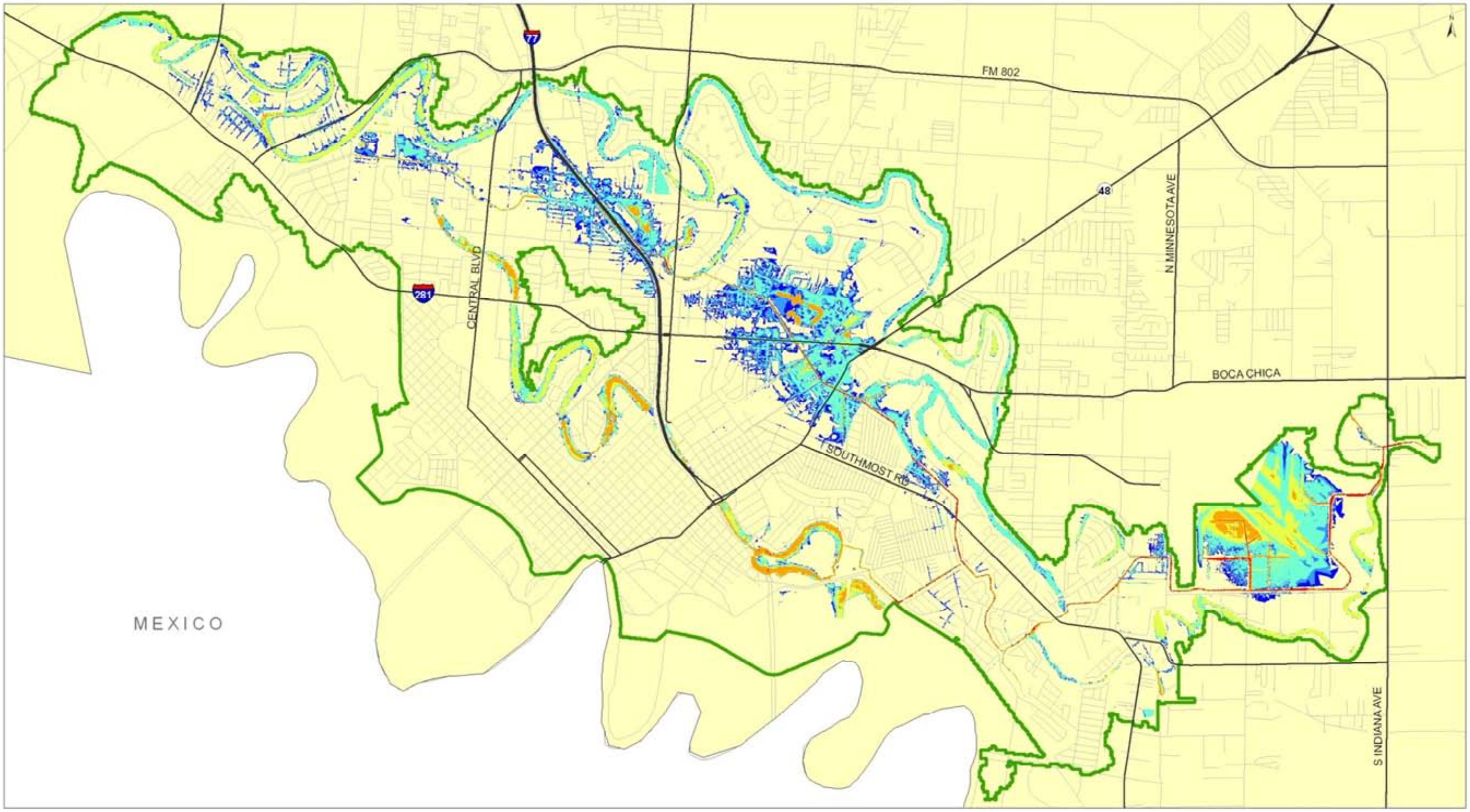
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development and Culvert Upgrades - 50-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

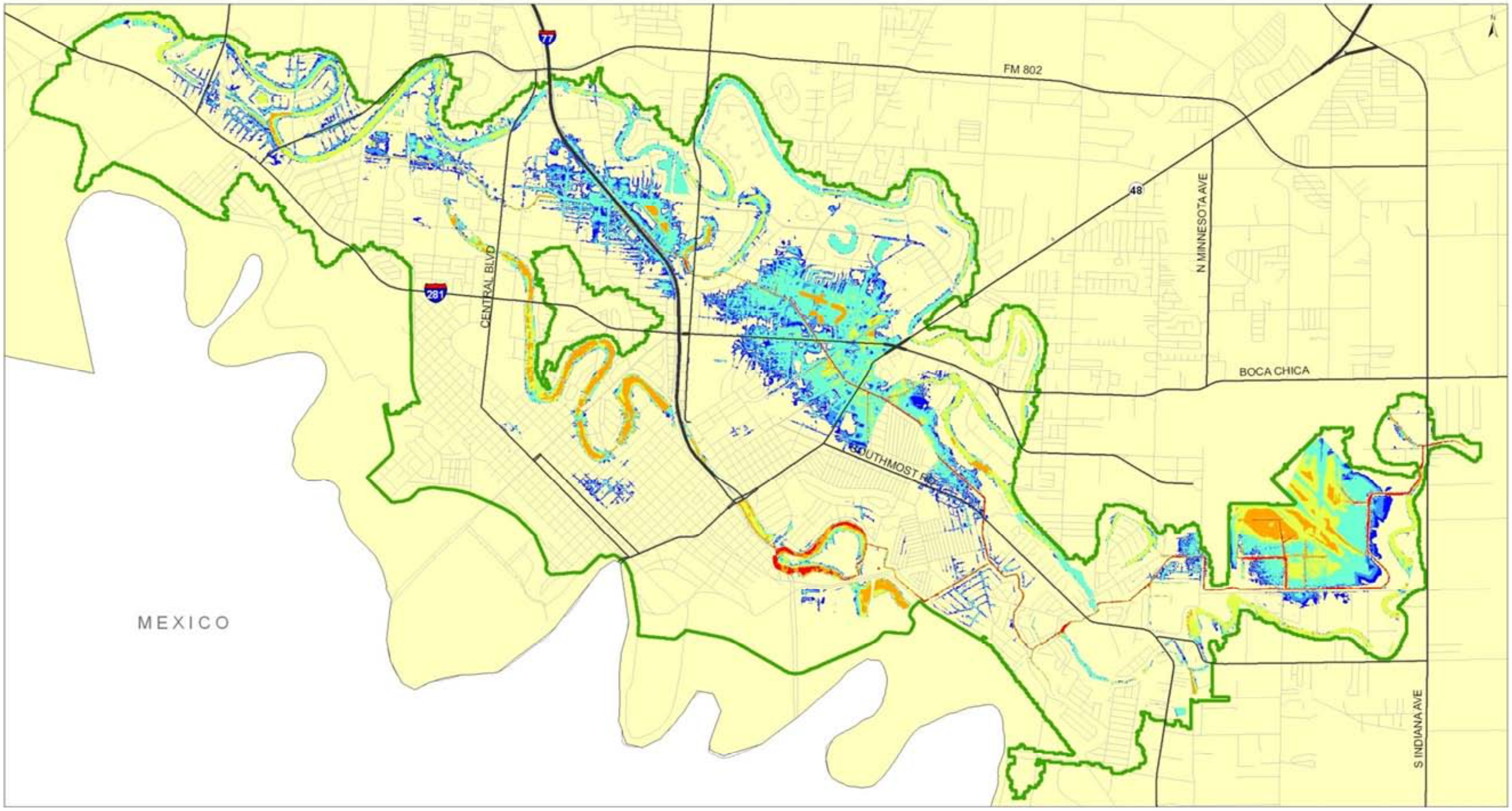
Watershed

1:20,000

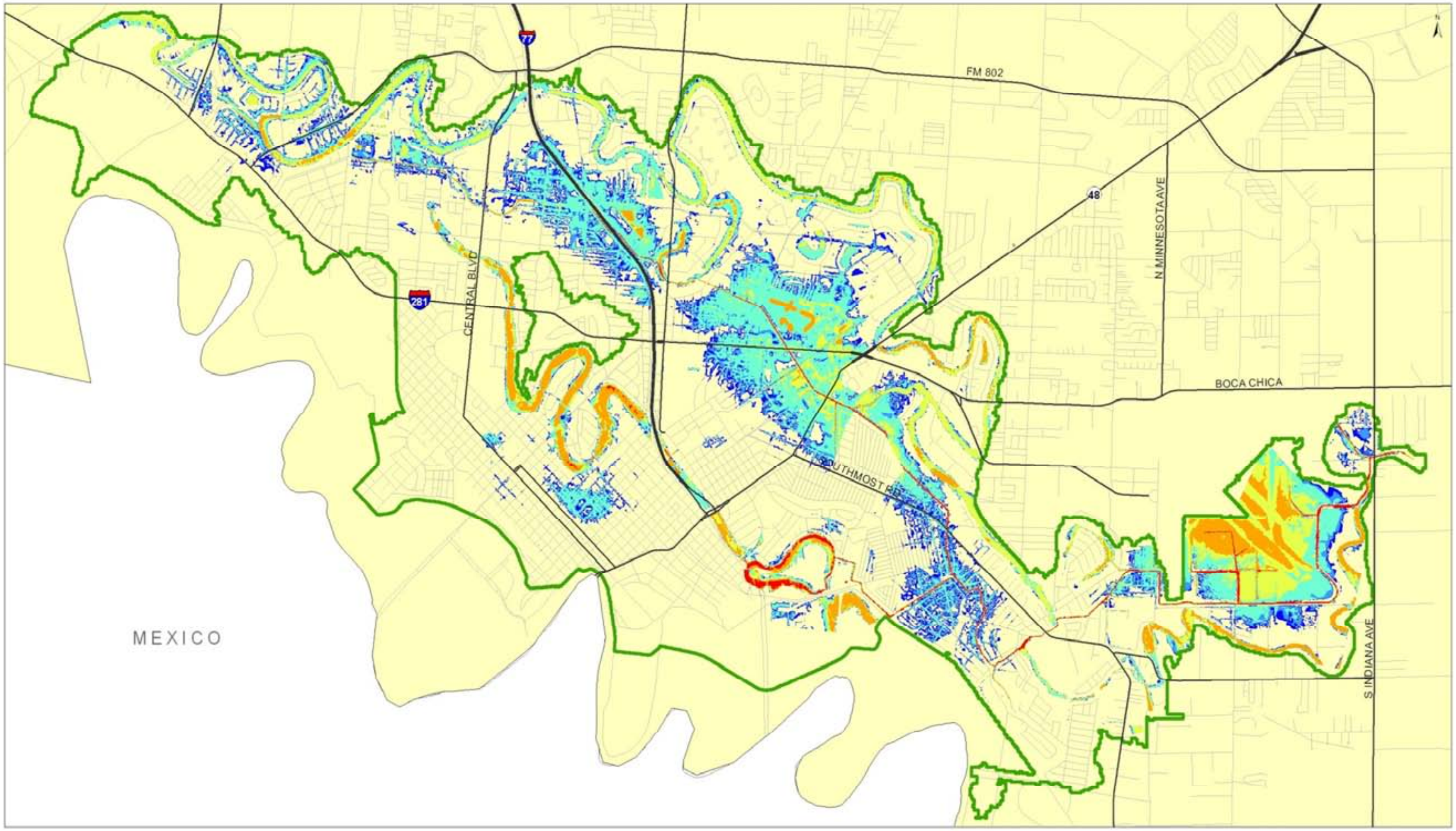
AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development, Diversion and Pump Doubling- 2-yr Storm Event



Flood Depth (feet)	Watershed	AMBIOTEC GROUP	City of Brownsville, Texas
0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10	1:20,000		Resaca de La Guerra, North Main Drain and Town Resaca Watersheds Existing Development, Diversion and Pump Doubling- 5-yr Storm Event



Flood Depth (feet)

0 - 0.5 | 0.5 - 1 | 1 - 3 | 3 - 5 | 5 - 10 | >10

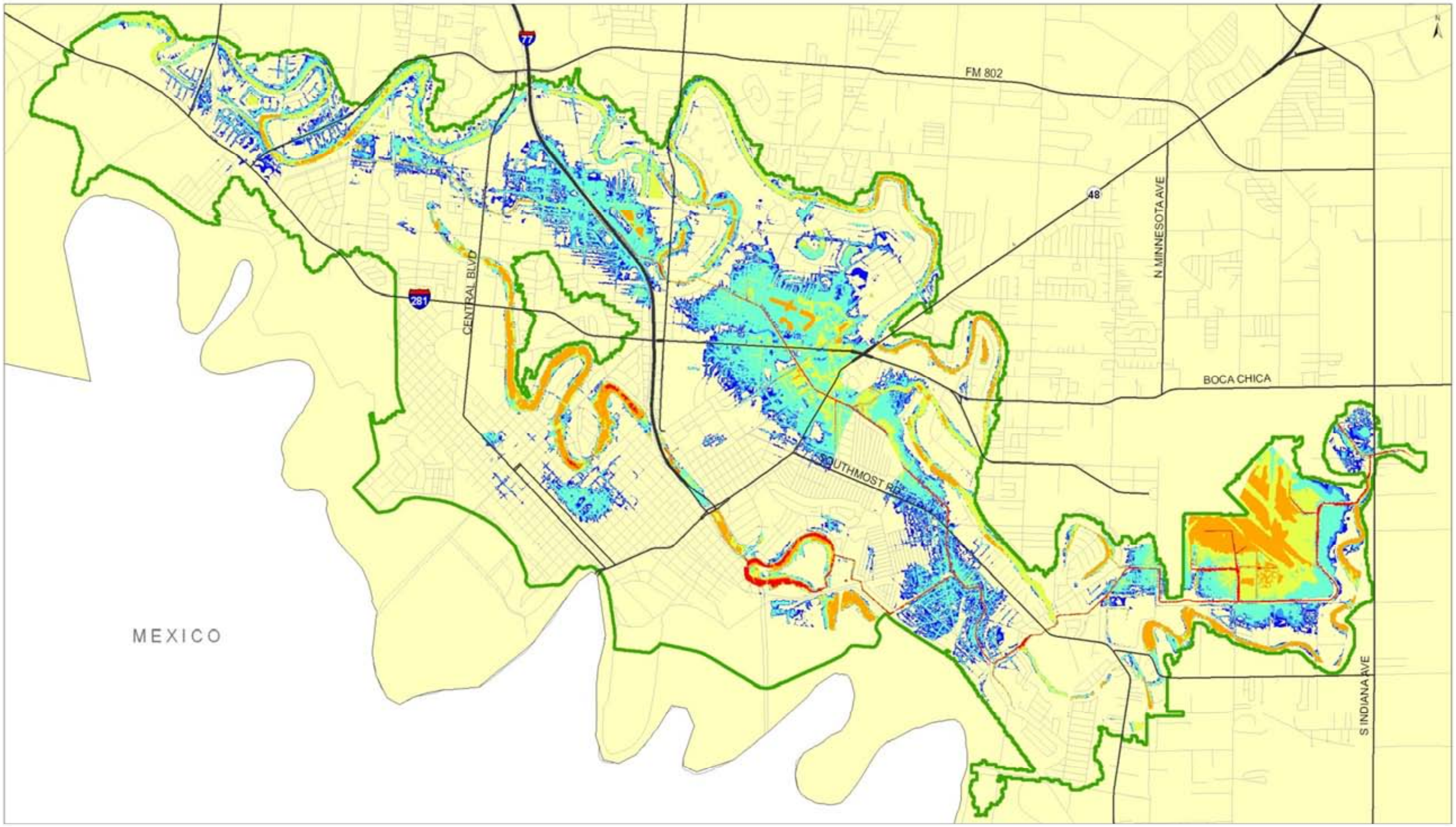
Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development, Diversion and Pump Doubling- 25-yr Storm Event



Flood Depth (feet)

0 - 0.5, 0.5 - 1, 1 - 3, 3 - 5, 5 - 10, >10

Watershed

1:20,000

AMBIOTEC GROUP

City of Brownsville, Texas

Resaca de La Guerra, North Main Drain and
Town Resaca Watersheds
Existing Development, Diversion and Pump Doubling- 50-yr Storm Event