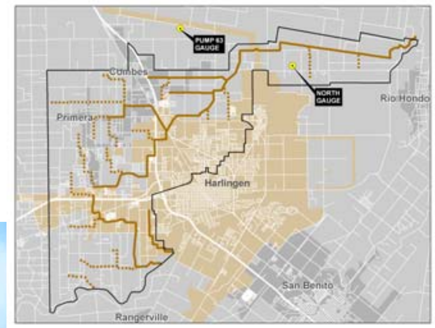
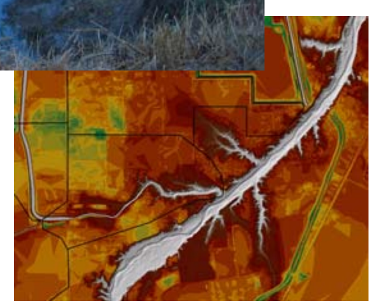
Espey Consultants, Inc.

CAMERON COUNTY DRAINAGE DISTRICT #5
Flood Protection Plan





In Association With:

Texas Water Development Board
Cameron County Drainage District #5
Cameron County
City of Harlingen
Towns of Primera, Palm Valley, and Combes

October 30, 2008

Project No. 6033

CAMERON COUNTY DRAINAGE DISTRICT #5 FLOOD PROTECTION PLAN
Engineering Report

Prepared for:

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TABLE OF CONTENTS

LIST OF APPENDICES

1.0 EXECUTIVE SUMMARY

1.1 SCOPE OF THE PROJECT

This document is a Flood Protection Plan for Cameron County Drainage District #5 (CCDD5), located in northern Cameron County, including portions of the communities of Harlingen, Primera, Palm Valley and Combes. The CCDD5, established in 1993, functions as a custodian of the drainage network in this area, responsible for construction, improvements, and maintenance of approximately 67 miles of open drains. In response to local concern over drainage problems and the need to approach the issues on a comprehensive, system-wide basis, CCDD5 and its community partners applied for funding assistance through the Flood Protection Planning Program of the Texas Water Development Board. The project was awarded funding in March of 2006 and contracts were executed in June of 2006.

The purpose of the project was develop a comprehensive set of models of the District's main drainage-way system, to be utilized in developing flood protection alternatives, both structural and non-structural. A set of policy goals and a corresponding implementation action plan were developed on the basis of the hydrologic and hydraulic models, Advisory Committee, and citizen input.

1.2 QUANTIFYING THE FLOODING ISSUES

This study included the development of a new hydrologic model (HEC-HMS) to estimate peak discharges at various points of interest throughout the District's ditch network. These peak discharges were determined for several different scenarios representing the flood risk for both present and future conditions. In terms of annual chance exceedance, the following frequency events were modeled: 1%, 4%, 10%, 20%, and 50% for existing and ultimate development conditions. Given the extremely flat topography of Cameron County, specific methods of predicting runoff were used (Kerby-Kirpich loss equations and the application of a non-standard peak rate factor) and refined through a calibration to observed rainfall and high water mark data following the May 25, 2007 event.

To determine the flooding extents and depths in the community, a series of hydraulic models were developed (HEC-RAS) to reflect the risk faced by the community in each of the modeled scenarios.

1.3 FLOOD PROBLEM AREAS

The analysis considered each of the four major drain networks maintained by the District: the North Main Drain, Stuart Place Main, Dixieland Main and Southwest Main. The North Main is the largest network, and also has the most problem areas. Along the main stem of the North Main Drain, reaches extending from Dilworth Road to New Combes Highway and from Breedlove Street to just downstream of FM 507 provide less than a 2-year level of service. The laterals draining the Town of Primera have only a 2-year level of service. Stuart Place Main, by contrast, generally has very good capacity. Along the Dixieland Main, the ditch upstream of Garrett Street poses a significant risk to flooding. The culvert at Bothwell Street obstructs flow at less than a 2-year event. This results in flooding as far downstream as Garrett Street. Southwest Main has low capacity upstream of Cook Lane.

1.4 FLOOD PROTECTION GOALS

To approach the complex issues of flooding in the District's service area comprehensively, a set of goals were established in the planning process to guide the District's decision making:

Goal 1: Proactively address flood problem areas with targeted improvements that consider the entire District's service area

Goal 2: Ensure that new development does not adversely affect property downstream

Goal 3: Upstream of the District's ditch network, local development should ensure positive drainage to the District's network; the District should ensure the lowest possible tailwater conditions to facilitate local drainage

Goal 4: Protect and enhance available storage in the system

Goal 5: Actively inform the community of the risk of flooding

Goal 6: Aggressively pursue a regional approach to curb illegal dumping

Goal 7: Update and refine the Flood Protection Plan on a bi-annual basis

1.5 ALTERNATIVES ANALYSIS AND RANKING

Each of the flood protection alternatives brings specific benefits and has identifiable costs associated with its implementation. To assess the economic viability of the alternatives, construction costs were calculated and the relative benefit of each under a 1% annual chance and 10% annual chance event was estimated using HAZUS-MH software. A Benefit-Cost Ratio (BCR) was then calculated. Based on the criteria of 1% annual chance benefit, 10% annual chance benefit, and BCR the alternatives were sorted and ranked, as follows:

Composite Scoring

PRIORITY	PROJECT
1	Alternative 10: Primera Improvements
2	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector
3	Alternative 3: Offline Detention Basin
4	Alternative 6: Proposed Channel Improvement
5	Alternative 2: Railroad Bridge Replacement

1.6 IMPLEMENTATION

Implementation of the Flood Protection Plan involves a multi-year, multi-faceted and phased approach that seeks to leverage partners in flood protection planning. The following table summarizes the Implementation Plan for CCDD5:

Action Item	Funding Source / Regulation / Effort Type	Priority
Action 1.1 : Complete construction of Alternative 1, Wilson Tract Diversion	Capital Budget	Immediate
Action 1.2 : Design and construct a detention pond at the borrow pit located south of FM 2994 and west of US 77 (Alternative 3)	Capital Budget; Hazard Mitigation Grant Funds	Immediate
Action 1.3 : Develop a multi-year implementation program to construct targeted channel and bridge improvements on the Nort Main Drain identified in Alternatives 6 and 7.	Capital Budget; Hazard Mitigation Grant Funds	Short-Term
Action 1.4 : Assist the Town of Primera in placing the identified improvements of Alternative 10 into their Comprehensive Plan, in order to leverage developer participation (see Item 3.1)	Capital Budget; Hazard Mitigation Grant Funds; Public-Private Partnerships; Impact Fees	Short-Term; Implement projects after Alternative 6 (Action 1.3)
Action 1.5 : Begin disussions with the railroad to cooperatively replace the skewed piers with box culverts (Alternative 2)	Capital Budget /both parties	Long-Term
Action 2.1 : Evaluate the feasibility of requiring on-site detention for at least the 10% annual chance event to mitigate the impacts of site-specific development	Joint task force of City, District, County, Towns	Immediate
Action 2.2 : Coordinate with local governments to establish common standards and hydrologic and hydraulic methods and assumptions	Joint task force of City, District, County, Towns	Immediate
Action 3.1 : Following the implementation of Action Item 1.3 (Alternative 6), begin a phased program to implement the recommended channel and culvert improvements of Alternative 10.	Capital Budget; Hazard Mitigation Grant Funds; Public-Private Partnerships; Impact Fees	Long-Term
Action 4.1 : Acquire (by right-of-way or easement) areas which are subject to high headwater conditions and where analysis indicates that increasing conveyance at that location will result in adverse downstream impacts.	Capital Budget / Dedication through development process	Short-Term
Action 4.2 : Acquire sufficient right-of-way to introduce a bench channel section in implementing Alternative 6, and in other areas where feasible.	Capital Budget	Immediate; Ongoing
Action 5.1 : The current level of risk for 100-year and 25-year events should be made freely available through dissemination of floodplain maps, both paper and digital.	CCDD#5 / City of Harlingen / Cameron County	Immediate
Action 5.2 : Make an initial presentaiton to the Chamber of Commerce, and follow up with annual update presentations, or contribute articles to the Chamber's newsletter with updates on CCDD#5 activities.	-	Long-Term; Ongoing
Action 5.3: Identify neighborhood leaders in flood-prone neighborhoods and develop a specific outreach campaign with their guidance.	-	Short-term; On-going
Action 5.4: Work with private inudstry and other stakeholders to develop and implement a program to distribute NOAA All Hazards Weather Radios to the public.	Public-Private Partnership; grant funding	Long-Term
Action 5.5: Working with other authorities, develop a specific plat note requirement to explain the limitations of flood protection in the Lower Rio Grande Valley.	Joint task force of City, District, County, Towns	Short-Term (together with 2.1, 2.2)
Action 6.1 : Recognizing that the illegal dumping problem is a regional issue, work with the Lower Rio Grande Valley Development Council (LRGVDC) to find the best long-term solutions	Multiple regional entities	Short-Term / On-going
Action 6.2 : Pursue grant funded opportunities through TCEQ and LRGVDC to host "clean-up" activities	Capital Budgets; TCEQ grant funding	Short-Term
Action 6.3 : Develop a public awareness program including signs, slogans, posters, etc.	Multiple regional entities	Short-Term
Action 6.4 : Install gates at access points	Capital Budgets	Long-Term
Action 6.5 : Reach out to neighborhood leaders to explain the issue and risks at-hand, and solicit their input on ways to curb the problem and raise awareness	-	Short-term; On-going (together with 5.3)
Action 7.1 : Continue the installation of telemtry-based gages toa monitor flow, stage, and velocity	Capital Budget; FEMA funding w/County and City of Harlingen	Long-Term
Action 7.2 : Perform a model update on a bi-annual basis to incorporate new development and calibration data, if available.	Capital Budget	Long-Term
Action 7.3 : Assess and prioritize the remaining construction projects, knowing that many conditions in the watershed will change over time	-	Long-Term; On-going

2.0 INTRODUCTION

2.1 BACKGROUND

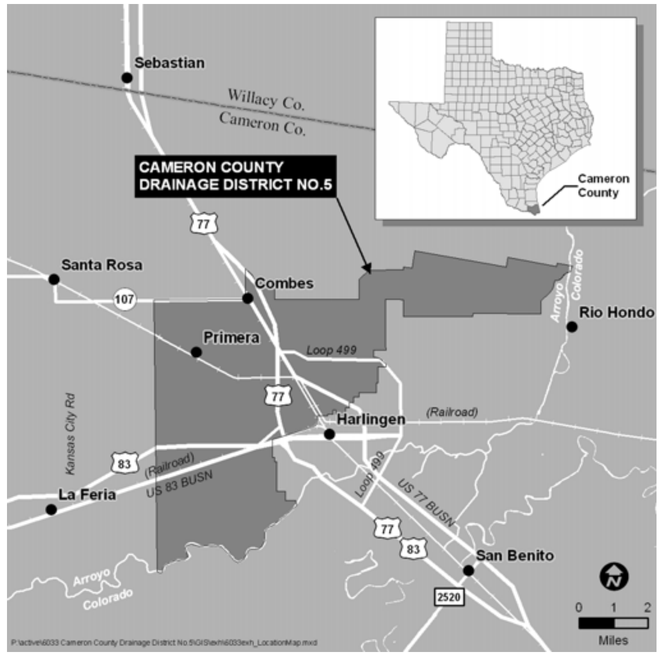


Figure 1. Location Map

Cameron County Drainage District #5 covers approximately 40 square miles within Cameron County and includes portions of the City of Harlingen and the Towns of Combes, Palm Valley, and Primera. The CCDD5, established in 1993, functions as a custodian of the drainage network in this area, responsible for construction, improvements, and maintenance of approximately 67 miles of open drains. Prior to 1993, the drainage ditches were the responsibility of Cameron County Irrigation District #1. Historically irrigation fees were based on water usage, so as farms were replaced by residential and commercial developments, irrigation and District revenues decreased while the magnitude and frequency of flood events increased. The Drainage District was formed to address this disparity, a need which became apparent after the flood of 1991. To date, $8.5 million have been spent on right-of-way acquisitions and improvements.

Recent development has caused an increase in the magnitude and frequency of flood events, and projected growth will continue to exacerbate the problem. As land use changes with development, more impervious cover increases the amount of rainfall runoff, leading to increased peak discharges. The communities in the Lower Rio Grande Valley have undergone significant change and growth within the last thirty years, transforming small, agricultural communities with acres of undeveloped land to suburban communities within the fourth-fastest growing metropolitan region in the State of Texas (Source: Texas State Data Center, 1990-2004 population data and estimates). This economic growth translates into increased development pressures on remaining, developable land within the CCDD5 jurisdictions. Older residential areas were often developed without consideration of upstream hydrology, downstream impacts, and before the adoption of the National Flood Insurance Plan (NFIP). Previous efforts in planning and capital improvement have been undertaken mostly in response to specific problem areas, rather than as a comprehensive watershed flood protection plan.

The Cameron County Drainage District #5 Flood Protection Plan (CCDD5 FPP) is made possible through the Texas Water Development Board Flood Prevention Planning Program, a program that offers grants to political subdivisions for the study and analysis of flooding hazards and development of flood mitigation measures in an effort for regional planning. Recipients of this grant are members of the National Flood Insurance Program and match funding.

2.2 OVERVIEW OF FLOODING PROBLEM

Cameron County is located in the Arroyo Colorado watershed. There are two major natural waterways in Cameron County, the Rio Grande, which acts as the county's southern boundary, and the Arroyo Colorado, which flows northeasterly across the county and south of Harlingen. In 1935, the Rio Grande floodway, a system of dams, levees, and channels, was completed to reduce the extent of flooding from the Rio Grande. This system, operated by the International Water and Boundary Commission (IWBC), partially diverts flow from the Rio Grande into the Main Floodway. West of Mercedes a divisor dike splits the Main Floodway flow between the Arroyo Colorado and the North Floodway. The divisor dike controls flow into the North Floodway and the Arroyo Colorado. As a result, flooding from the Arroyo Colorado is not considered a risk to the City of Harlingen and the adjacent towns in the CCDD5 jurisdiction.

Cameron County, located along the Gulf Coast, can be subjected to intense rainfalls from thunderstorms and tropical depressions. The climate is sub-tropical and semi-arid, with an average annual rainfall of 26 inches. These intense rains provide a significant potential for flooding. Slowly permeable loamy and clay soils prevalent in this county and limited grade provide poor drainage. The table below lists some historical rainfall events in Cameron County (Slade, 2003).

Table 1. Historical Rain Events

Date	Description	Max Depth (in.)	Death and Damages
May 25, 2007	Excessive rain and flash flooding; 9-12 inches of rainfall within 24 hours	12	Unknown
September 16-18, 1988	Rainfall produced by Hurricane Gilbert	6.4	Minor damage reported in Texas: beach erosion and tornados; 327 deaths, mostly in Mexico; Total damage estimated at $5.5 billion
February 6, 1987	Torrential rains of 6-7 in. fell during a 2-hour period in parts of Brownsville in Cameron County.	7	Unknown
September 16-19, 1984	Heavy rains, some exceeding 20 in., drenched the lower Rio Grande Valley.	20	Worst flooding for Cameron County since Hurricane Beulah; Approximately 50 percent of the eastern Cameron County flooded
February 18-21, 1982	Storms dumped 6 in of rain in less than 3 hours at Harlingen	7.42	Estimated $250,000 in Cameron County
September 15-25, 1967	breaking magnitude on many streams southern Texas and northeastern Mexico. Estimated 34 in.of rainfall on the Nueces River Basin.	34	44 deaths total, 15 deaths in Texas: Estimated $100 million

Source: USGS Website Major and Catastrophic Storms and Floods in Texas

The analysis in the CCDD5 FPP is concerned with the hydraulic capacity of main channels of the drainage ditches. This study does not analyze localized flooding issues.

2.3 PROJECT SCOPE

The purpose of this project is to identify flooding issues in the CCDD5 drainage system and provide mitigation alternatives. The following tasks and public input components were performed in this study:

- Conduct an initial kick-off meeting with an appointed Advisory Committee

The Advisory Committee consisting of representatives of the participating entities and members of the CCDD5 Board of Directors, met on October 19, 2006. The project schedule and responsibilities of participants were set. A public meeting was held that evening at Primera Town Hall, to provide an overview of the project scope and receive citizen input on the scope and their flooding issues. Property flood history forms, designed to solicit quick input on the nature and extent of the flood problems, were made available at this meeting.

- Data collection and review of flood and drainage problem areas

Flood-prone areas were identified based on citizen input and records. Available GIS datasets, current and future land use maps, soil maps, cultural resource maps and materials, environmental resource maps and materials, LIDAR topography, digital orthophotography, cross-section data, existing FEMA models, and previous drainage, engineering, and geotechnical studies were assembled by the District and its contract for base map creation. Information on previously identified critical environmental features was also obtained. The gathered information was reviewed. Flood prone areas were classified according to primary drainage system problems and secondary drainage system problems. The specific recommended problem areas for study were identified. Environmental constraints were researched and reviewed to identify possible critical environmental features that may need to be considered during alternative development.

- Collect field survey

A list of required field survey data was compiled identifying critical bridges and culverts and channel cross-sections. Sixty-seven culverts and bridges and 102 cross sections were surveyed for this study.

- Develop hydrologic models

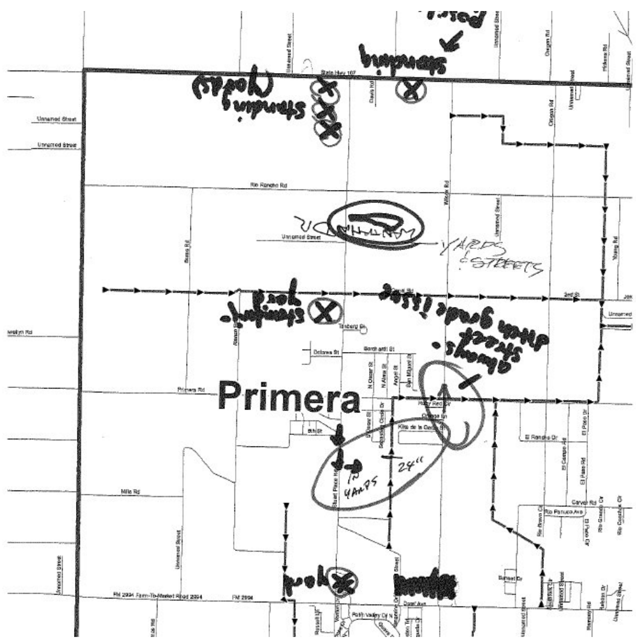
CCDD5 was divided into 59 subbasins to model the four sub-watersheds: North Main Drain, Wilson Main Drain, Stuart Main Drain, and Southwest Main Drain. Existing GIS coverages of the City of Harlingen and Cameron County was analyzed in ArcGIS 9.2 to develop hydrologic parameters. The 50%, 20%, 10%, 4%, and 1% annual chance storm events and the ultimate conditions 1% annual chance event peak flow rates were developed with HEC-HMS.

- Develop hydraulic model

HEC-RAS was used to model the primary drainage ditches and laterals in CCDD5. The HEC-RAS models were improved with collected field survey data and information from design plans. Hydraulic analyses were performed to evaluate the existing conditions 50%, 10%, 4%, and 1% annual chance storm events and the ultimate conditions 1% annual chance event. Floodplain maps for the 50%, 20%, 10%, 4%, and 1% annual chance storm events and the ultimate conditions 1% annual chance event were developed.

- Present initial findings at second Public Meeting

Figure 2: Citizen Input from June 12, 2007 Meeting



Based on review of the gathered information and initial modeling efforts, a preliminary summary of methodology and modeling approach was prepared and presented at the public meeting held on June 12, 2007. Results with the preliminary hydrology and hydraulic calibrations with the May 25, 2007 rainfall event were presented, as well as the next steps to be taken toward completing the floodplain protection plan. Citizens were asked to draw the extent and nature of the flooding problems they experienced on a map to help distinguish between localized flooding issue areas and riverine flooding issues. An excerpt from this map is included here as Figure 2.

- Review flood protection criteria and develop, analyze, and prioritize mitigation alternatives

Based on a review of existing design flood criteria and determination of acceptable level of flood protection with focus on problem areas, structural and non-structural flood control measures were developed. A cost-benefit analysis was performed for each alternative. Results of the Benefit-Cost Analysis were discussed and alternatives prioritized at an advisory committee meeting on Tuesday, October 23, 2007.

- Develop plan for implementation and phasing

A plan with recommendations for the implementation and phasing of the improvements was developed. The implementation plan identifies potential funding sources for the improvements and coordinates with the CCDD5 current Capital Improvements Plan.

- Prepare final flood protection plan

A final plan was prepared and presented at a final public meeting on November 29, 2007. This document and attachments represent the final deliverables. The deliverables include maps, technical analysis and supporting documentation, and the goals/action, implementation and phasing plan.

The study provided in this Flood Protection Plan does not duplicate the FEMA re-study. The Federal Emergency Management Agency, under the Map Modernization program, initially committed nearly $2M to update floodplain maps in Cameron County in FY2005 and FY2006. However, while the re-study effort will map portions of the Arroyo Colorado, there are no segments within CCDD5 which are included in the FEMA re-study. Thus, the CCDD5 FPP study complements the FEMA work. Furthermore, the CCDD5 FPP provides better detail than the limited detail studies proposed in this part of Cameron County. For example, a more detailed and accurate rainfall-runoff model (HEC-HMS) was developed, while the FEMA re-study proposes to only use adjusted regression equations. For some of the smaller communities, such as Palm Valley, Combes and Primera, which are not included in the current FIS or FEMA re-study, funding under the Flood Protection Planning program was the only means to accomplish floodplain management and planning.

2.4 PREVIOUS FLOOD STUDIES

Several studies have been completed in the Cameron County area. These studies include Flood Insurance Studies (FIS) performed by the Federal Emergency Management Agency (FEMA) and a Feasibility Study for Cameron County performed by the United States Army Corps of Engineers (USACE):

FEMA FIS 1999 Unincorporated Areas of Cameron County, Texas
The study area included southeast portions of Cameron County. Three principal waterways in the county were studied including the Rio Grande, North Floodway and the Arroyo Colorado Floodway. None of the area studied in the CCDD5 FPP is included in this study.

FEMA FIS 1981 City of Harlingen, Texas
The study area includes the incorporated area of the City of Harlingen. The streams selected for detail study were the Arroyo Colorado, and three tributaries to the Arroyo Colorado. None of the studied areas are included in the CCDD5 FPP.

FEMA FIS 1980 City of San Benito, Texas
The study area includes the incorporated area of the City of San Benito. San Benito borders Harlingen on the southwest. None of the studied areas are included in the CCDD5 FPP.

USACE 1990 Feasibility Study of Cameron County, Texas
This study was done to determine the feasibility of Federal participation in flood control measures to reduce flood damages in Cameron County. This study analyzes the Arroyo Colorado, the North Floodway, and the

Main Floodway. Of the channels studied in the CCDD5 FPP, only portions of North Main Drain are analyzed in detail in the 1990 Feasibility study. Copies of the models could not be located, and digital data was not available from the USACE.

2.5 LOCATION AND DESCRIPTION OF WATERSHED

The CCDD5 watershed is approximately 43 square miles. The district boundaries generally represent the limits of the watershed. The natural topography of CCDD5 is typical of the Rio Grande Delta Plain with mildly sloping terrain. The elevations vary from approximately 50 ft near the southwest quadrant of CCDD5 to 25 ft near its outfall into the Arroyo Colorado. The terrain slopes at approximately 1 foot per mile. Generally, north of US 83, CCDD5 slopes northeasterly, with the exception of Primera and Combes, which slope towards the south. South of US 83, CCDD5 generally slopes towards the south. The primary drainage system is provided by a network of man-made channels. A brief description and diagram of the four main sub-watersheds and their respective network follows:

North Main Drain

The North Main watershed drains approximately 33 square miles. The North Main Drain is the principal drain which services Harlingen, Primera, Palm Valley, and Combes. The main stem initially drains the southern half of Primera, continues south through the town of Palm Valley, runs easterly and northeasterly through Harlingen and along the north edge of the city before it outfalls into the Arroyo Colorado.

Stuart Place Main

The Stuart Place watershed drains approximately 8 square miles. The Stuart Place Main network directs water from the Palm Valley area south to the Arroyo Colorado, a distance of approximately four miles. The ditch begins south of Bougainvillea Drive. Flow travels south and easterly before it outfalls into the Arroyo Colorado, south of the Dixieland Reservoir. The Acacia channel connects North Main Drain and Stuart Place Main, allowing flow diversions depending on channel conditions.

Dixieland Main

The Dixieland Main watershed drains approximate 1 square mile. The Dixieland Main network begins south of Lincoln Road. Flow travels south, parallel with Dixieland Road and continues south and easterly before it outfalls into the Arroyo Colorado, south of the Tony Butler Golf Course.

Southwest Main

The Southwest Main watershed drains approximately 1 square mile. This network serves the area southwest of the Stuart Place Main. Flow travels easterly and southerly from Baker Potts Road through the Stuart Place County Estates Subdivisions with an outfall into the Arroyo Colorado.

The figure below illustrates the location of CCDD5 area relative to the Arroyo Colorado and the four sub-watersheds and respective ditch networks.

The secondary drainage system includes minor ditches, storm sewer systems and roadway gutters. The hydraulics of the secondary system is not analyzed.

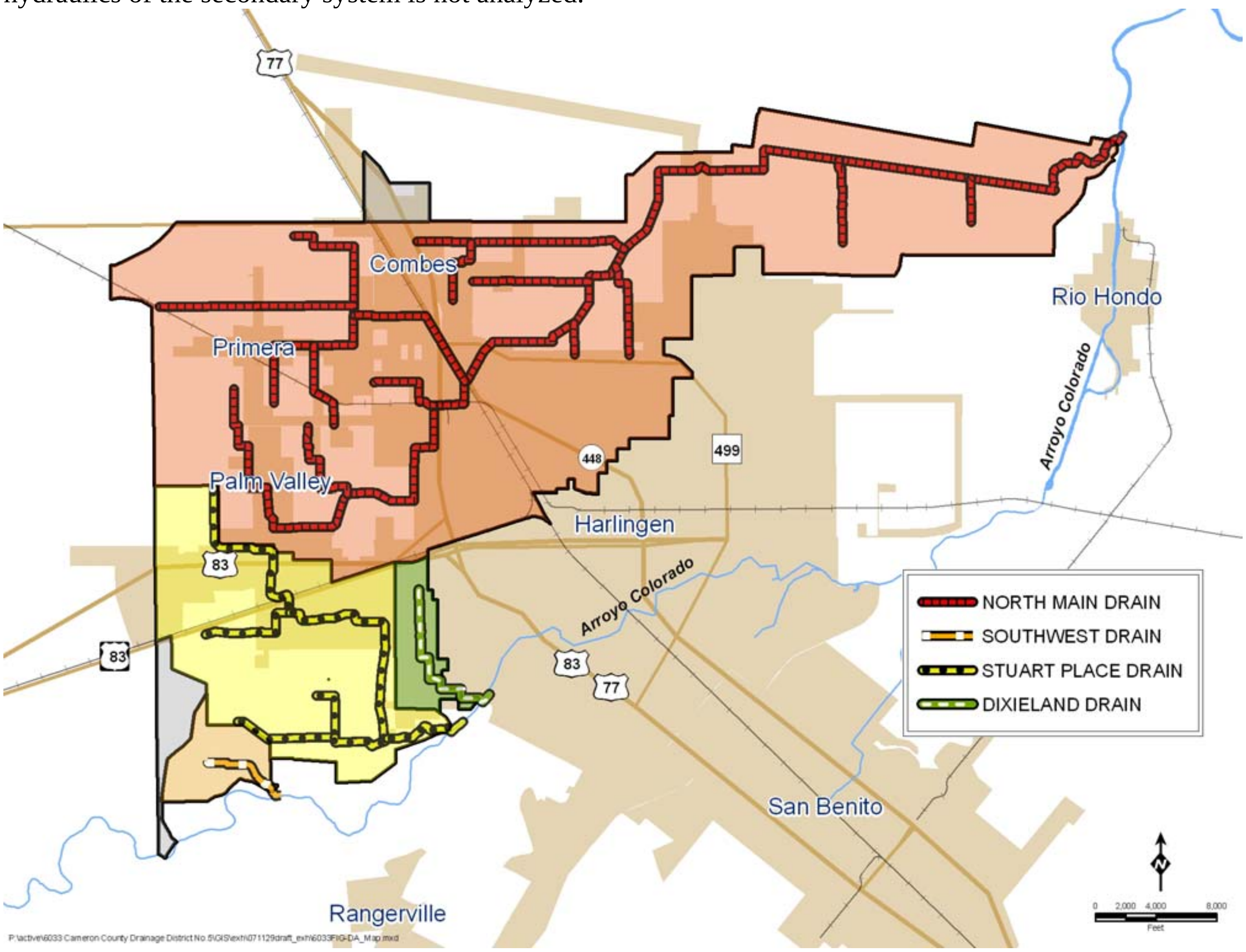


Figure 3. Cameron County Drainage District #5

3.0 HYDROLOGIC ANALYSIS

The scope of this project includes a hydrologic study of CCDD5. The hydrologic analysis includes the evaluation of the existing conditions 50%, 20%, 10%, 4%, and 1% (2-, 5-, 10-, 25-, and 100-year) annual chance storm events. The hydrologic analysis also evaluates the ultimate conditions 1% annual chance event.

Version 3.1.0 of the HEC-HMS computer program developed by the Hydrologic Engineering Center of the U. S. Army Corps of Engineers (USACE) is used in this analysis to estimate peak flow rates along each reach. Peak flow rates are computed along the watercourses for the existing 50%, 20%, 10%, 4%, 1%, and ultimate 1% annual chance storm events. This hydrology section describes the input parameters used in this analysis, the calibration efforts, the correlation with frequency analyses, and the computed peak flow rates to be used in the floodplain analysis.

3.1 DRAINAGE AREA DELINEATION

The CCDD5 watershed was divided into fifty-nine subbasins using United States Geological Survey (USGS) topographical survey data, aerial photography, LIDAR data, field visits, and the 1990 U.S. Army Corps of Engineers Feasibility Report for Cameron County. Fifty-seven of the subbasins drain into one of the four main subwatersheds, described in Section 1.5, and ultimately drain to the Arroyo Colorado. The other two subbasins were not tributary to modeled streams. The drainage area map is included in Appendix A as Exhibit 1.

3.2 PRECIPITATION

The precipitation depths are taken from a USGS publication by Asquith and Roussel, *Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas*, 2004. Table 2 below shows the precipitation depths for various durations for the studied events.

Table 2. USGS Storm Depths for the CCDD5 Flood Protection Plan

		USGS Cumulative Depth (in)				
Time	Time (min)	50%	20%	10%	4%	1%
15 minutes	15	1.0	1.4	1.6	1.9	2.3
1 hour	60	1.9	2.5	3.0	3.5	4.5
2 hours	120	2.4	3.1	3.7	4.5	5.8
3 hours	180	2.5	3.3	3.9	4.6	6.2
6 hours	360	2.8	3.8	4.6	5.7	8.0
12 hours	720	3.2	4.4	5.3	6.5	9.0
24 hours	1440	3.5	5.1	6.0	7.5	10.0
48 hours	2880	4.1	6.0	7.0	9.0	12.0

3.3 INFILTRATION LOSSES

The U.S. Department of Agriculture Natural Resource Conservation Service (NRCS, formerly the Soil Conservation Service, SCS) has developed a rainfall runoff index called the runoff curve number (CN). The runoff curve number takes into account such factors as soil characteristics, land use/land condition, and antecedent soil moisture. This number is used to derive a generalized rainfall/runoff relationship for a given area. A description of these components and the equations for calculating runoff depth from rainfall are provided below.

The NRCS classifies soils into four hydrologic soil groups: A, B, C, and D. These groups indicate the runoff potential of a soil, ranging from a low runoff potential (group A) to a high runoff potential (group D). Digital soil data is available from the Texas Natural Resource Information System (TNRIS) post-processed from the US Department of Agriculture Soil Survey Geographic (SSURGO) database into the Texas statewide mapping system. A map of the soils found in CCDD5 is included as Exhibit 2 in Appendix A.

The NRCS provides runoff curve numbers for three Antecedent Moisture Conditions (AMC): I, II and III. AMC I represents dry soil conditions and AMC III represents saturated soil conditions. AMC I is used for areas that have the lowest runoff potential. In general, AMC II is considered to be the typical soil condition; however, studies have indicated that AMC II is not appropriate in all parts of Texas. Investigations have shown that the average condition ranges from AMC I in west Texas to between AMC II and III for east Texas. Runoff curve numbers vary from 0 to 100, with the smaller values representing soils with lower runoff potential and the larger values representing soils with higher runoff potential. This study assumes an AMC I to represent average condition as shown in Figure 3. AMC was also used as a calibration parameter as described in Section 4.0.

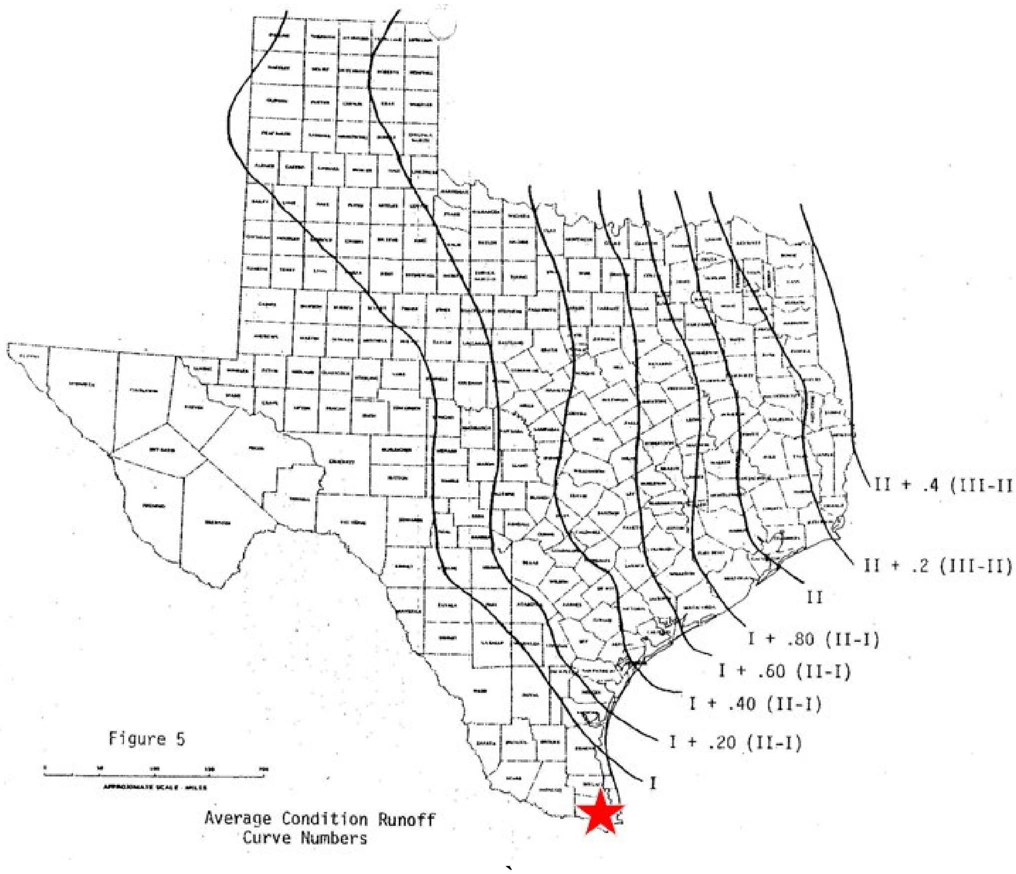


Figure 4: Antecedent Moisture Condition Determination
Source: SCS Technical Note *Estimating Runoff for Conservation Practices*

Curve numbers (CN) were evaluated independent of imperious cover (i.e., these curve numbers reflect fair conditions, open spaces, brush cover) for this analysis. The table below lists the CN values for CCDD5.

Table 3. NRCS Curve Number Table

Curve Numbers (CN)			
Soil Group	AMC I	AMC II	AMC III
A	19	35	55
B	36	56	75
C	51	70	84
D	59	77	89

Key Assumptions: brush cover type, fair hydrologic condition
Source: TR-55

A composite CN is computed based on area weighting of each hydrologic soil group within each subbasin. Impervious cover values are entered separately from CN values into the HEC-HMS model. Calculations of the weighted curve number values for each subbasin are included in Appendix B. Weighted CN values under AMC I conditions were used for analysis.

HEC-HMS computes 100 percent runoff from impervious areas, while runoff from pervious areas is computed using the selected CN value and the following equations:

$$Q = (P - 0.2 \times S)^2 / (P + 0.8 \times S) \qquad \text{Equation 1}$$

And

$$CN = 1000 / (10 + S) \qquad \text{Equation 2}$$

Where:

Q	=	depth of runoff (in),
P	=	depth of precipitation (in),
S	=	potential maximum retention after runoff begins (in)[1], and
CN	=	runoff curve number.

3.3.1 Existing Impervious Cover Determination

Existing land use data was not available for the entire district. Impervious cover was determined using a spectral analysis of 2004 infrared aerial photography and updated with 2006 true color aerial photographs to reflect significant post-2004 development. Spectral analysis is the process of analyzing geological formations by interpreting the variation in color values from imagery. Through spectral analysis, vegetative cover was identified through its reflection of infrared light. Vegetative cover often constitutes the majority of pervious cover in a given study area. Areas not identified as pervious are assumed impervious, which typically consists of buildings and paved surfaces.

Small areas not previously identified as vegetation and pervious, due to shadows cast by higher trees or brushes, as well as occasional bare, unpaved ground spots, were significantly minimized through a cell neighborhood analysis. This step assured that the reversal output that yields impervious cover would not consist of pervious cover that was not covered by vegetation, as is often the case with bare ground spots, harvested crop fields, or sites of new development where land has been cleared and the area to be paved versus covered in grass. The existing impervious cover map is included in Appendix A as Exhibit 3.

3.3.2 Ultimate Impervious Cover Determination

The impervious cover values for each subbasin of CCDD5 were modified to reflect the projected ultimate land use assuming a 30-year planning horizon. Data sources used to extrapolate development conditions include the City of Harlingen Future Land Use Map and the City of Harlingen Zoning Ordinance (Section 7.01 (D) Lot Coverage). Residential area densities are provided by the Towns of Primera, Combes, and Palm Valley for areas outside Harlingen.

Harlingen's future land use map gives projected land use patterns while the zoning ordinance provides guidance on allowable impervious cover for each type of zoning district. A logical relationship was then established between the land use categories and zoning districts to derive an allowable ultimate impervious cover percentage. This percentage was reduced by 10% for each category to allow for landscaping and setback requirements found in the City of Harlingen Zoning Ordinance and to more accurately reflect realistic growth patterns. To determine ultimate impervious cover for the Towns of Primera, Combes, and Palm Valley, a dwelling units per acre (DUA) density description was assigned to all residential areas. For areas outside municipalities, an increase in impervious cover of 10% over existing was used to represent developed conditions at the end of the 30-year planning horizon. The following tables list the assigned impervious cover percent for each land use category found within CCDD5.

Table 4. Ultimate Impervious Cover Percentages for Harlingen and ETJ

LU CODE	DESCRIPTION	I.C. %
ROW	Transportation Right of Way	90%
RE	Retail	80%
GR	General Retail	90%
IN	Industrial	80%
LI	Light Industrial	90%
R1	Residential, Single Family	50%
R2	Residential, Duplex	50%
R3	Residential, Multi-Family	70%

Note: Impervious cover assumptions derived from a combination of the adopted Future Land Use plan found in Harlingen's Vision 2020 Comprehensive Plan and impervious lot coverage allowed under the zoning ordinances.

Table 5. Ultimate Impervious Cover Percentages for Primera, Combes and Palm Valley

LU CODE	DESCRIPTION	I.C. %
ROW	Transportation Right of Way	90%
RE	Retail	80%
GR	General Retail	90%
IN	Industrial	80%
LI	Light Industrial	90%
DUA	**DESCRIPTION**	**I.C. %**
0.5	Residential	4%
2	Residential	16%
3	Residential	24%
4	Residential	32%
4.5	Residential	36%
5	Residential	40%
6	Residential	48%
10	Residential	80%
12	Residential	96%

Note: Residential density derived from meetings with town officials.

The ultimate impervious cover map is included in Appendix A as Exhibit 4. A summary comparing existing and ultimate conditions impervious cover percentages is included in Appendix C.

3.4 UNIT HYDROGRAPH

A rainfall/runoff transformation is required to convert rainfall excess (total rainfall minus infiltration losses) into runoff from a particular subarea. Runoff hydrographs were generated for each defined subarea within the studied watershed. The unit hydrograph method represents a hydrograph for one unit [inch] of direct runoff and is a nationally accepted, standard engineering practice approach. Hydrographs were calculated using the user specified NRCS unit hydrograph and modified unit hydrograph methods. The following sections present each method. Further description of the unit hydrograph method with respect to calibration is described in Section 4.0.

3.4.1 User Specified Unit Hydrograph

The user specified unit hydrograph is a dimensionless unit hydrograph that incorporates a calculated peak discharge and time to peak. The dimensionless unit hydrograph developed by the NRCS, shown in Figure 4, was developed by Victor Mockus and presented in *National Engineering Handbook, Section 4, Hydrology*. The dimensionless unit hydrograph has its ordinate values expressed as a dimensionless ratio of discharge at time t to peak discharge, q/Qp, and its abscissa values as a dimensionless ratio time t and time to peak, t/Tp.

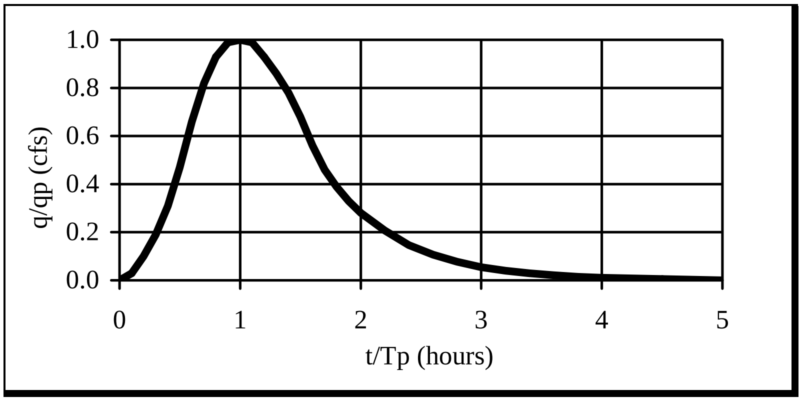


Figure 5. NRCS Standard Unit Graph

The user specified unit hydrograph requires of two input parameters, T_{LAG} and drainage area, A. T_{LAG} is the time between the center of mass of rainfall excess and the peak of the unit hydrograph (NRCS 1985). Lag is this delay in time after a rain event before the runoff reaches its maximum peak.

The time to peak is computed using the following equation:

$$T_P = \Delta t/2 + T_{LAG} \qquad \text{Equation 3}$$

Where:

T_P	=	time to peak of the unit graph (hours),
Δt	=	computation interval or duration of unit excess (hours), and
T_{LAG}	=	watershed lag (hours).

The peak flow rate of the standard NRCS unit graph is computed using the following equation:

$$Q_p = \text{PRF}*\text{A}/T_P \qquad \text{Equation 4}$$

Where:

Q_p	=	peak flow rate of the unit graph (cubic feet per second [cfs] / inch) and
A	=	watershed area (square miles).
PRF	=	peak rate factor (dimensionless)

Standard engineering practice uses a peak rate factor (PRF) of 484.

3.4.2 Modified NRCS Unit Hydrograph

Research in the paper *Revisit of NRCS Unit Hydrograph Procedures* (ASCE, 2005) examines the role of the peak rate factor (PRF) in the NRCS unit hydrograph method. This paper notes that the PRF value is correlated with the watershed's basin shape factor, which is defined as the drainage area divided by the square of the main channel length. Variability in shape factor implies variability in peak factor rate, but use of PFR value other than 484 will not maintain a unit hydrograph. This contradiction led researchers to examine an alternate method to develop a regional unit hydrograph based on a Gamma function. Research, delivered in TxDOT's 2005 paper *Time-Parameter Estimation for Applicable Texas Watersheds,* was based on data from 1600 rainfall-runoff data sets for 90 USGS gage stations in central Texas watersheds.

The paper provides a two-parameter fitted Gamma based unit hydrograph in which the PRF reflects the watershed's topography. PRF values may vary from 600 for steep terrain to 100 for very flat terrain. After selection of the peak factor rate, the parameters $\phi(\alpha)$ and α are calculated based on the following equations:

$$\phi(\alpha) = (T_p Q_p)/(645.33A) = PRF/645.33 \qquad \text{Equation 5}$$

and

$$\alpha = 5.53\,\phi(\alpha)^{1.75} + 0.04 \qquad \text{for } 0.01 < \phi(\alpha) < 0.35 \qquad \text{Equation 6}$$

$$\alpha = 6.29\,\phi(\alpha)^{1.998} + 0.157 \qquad \text{for } \phi(\alpha) > 0.35$$

The table below lists a description of terrain and their corresponding PRF with values of phi and alpha based on the Equations 5 and 6.

Table 6. PRF Values for Texas Watersheds (Gamma-based Unit Hydrograph)

PRF	Description	Φ	α
100	Very Flat	0.15	0.26
200		0.31	0.80
300	Flat	0.46	1.52
370	Texas Mean PRF	0.57	2.23
400		0.62	2.58
484	Standard NRCS PRF	0.75	3.70
500		0.77	3.94
600	Steep, Mountain Terrain	0.93	5.60

Based on Fang et al, ASCE 2005

The ordinates of the gamma hydrograph are discharge Q and time t. The discharge at a given time t are computed using the following equation:

$$Q = Q_p * \left(\frac{t}{T_P}\right)^{\alpha * \exp\left(\frac{1-t}{Tp} * \alpha\right)} \qquad \text{Equation 7}$$

Where:

Q	=	flow rate (cfs) at time t
T_P	=	time to peak of the unit graph (hours),
Q_p	=	peak flow rate of the unit graph (cubic feet per second [cfs] / inch)

t	=	time (hours)
α	=	parameter based on chosen PRF

The figure below compares two hydrographs with the same drainage area and time of concentration but different PRFs. The hydrograph with the PRF of 200 has a lower peak discharge and less sharp decline of the receding limb than that with a standard PRF of 484. The flat terrain introduces unique challenges related to hydrograph timing, which is better accounted for with a PRF of 200. The volume of both hydrographs is the same.

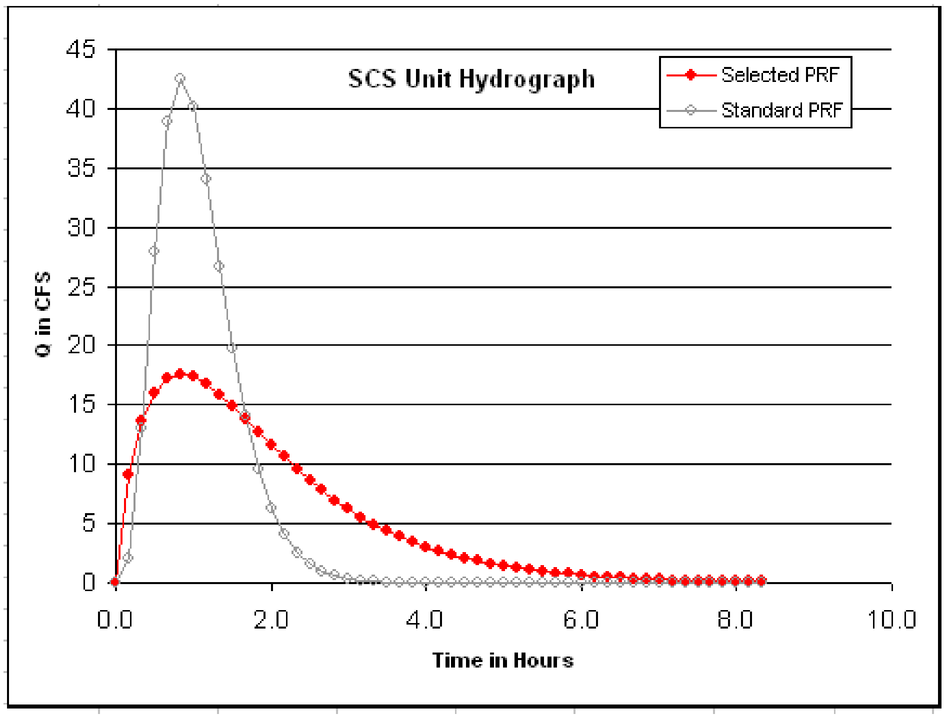


Figure 6. SCS Unit Hydrograph, Standard PRF = 484 v. Selected PRF =200

3.5 TIME OF CONCENTRATION

The time of concentration, Tc, is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed (NRCS 1985). Typically, the time of concentration may be estimated by calculating and summing the travel time for each subreach defined by the flow type. The method of calculating the time of concentration Tc was a factor in the calibration of the model. The Kerby-Kirpich and TR-55 methods were tested. The Kerby-Kirpich method was used and validated in calibrations runs. The following sections present each method. Further description of the time of concentration with respect to model calibration is described in Section 4.0.

3.5.1 TR-55 Method

The NRCS method assumes that the lag time of a watershed is 60 percent of the watershed's time of concentration. The time of concentration may be estimated by calculating and summing the travel time for each subreach defined by the flow type: sheet flow, shallow concentrated flow, and channelized flow (including roadways, storm sewers, and natural/manmade channels). The methods prescribed in the NRCS' Technical Release 55 (TR55) are used to determine the times of concentration for each flow segment in this analysis. Appendix D shows the results of the calculations for this analysis utilizing each typical flow segment presented below.

Sheet Flow (≤ 300 feet)

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact, of drag over the plane surface and obstacles such as litter, crop ridges, and rocks, and of erosion and transportation of sediment. These n values are for very shallow flow depths of approximately 0.1 foot. Assuming sheet flow of less than or equal to 300 feet, travel time is computed as follows:

$$T_T = (0.007 * (n*L)\ 0.8) / (P20.5 * s0.4) \qquad \text{Equation 8}$$

Where:

T_T	=	travel time (hr),
n	=	Manning's roughness coefficient,
L	=	flow length (ft),
P2	=	2-year, 24-hour rainfall (in), and
s	=	slope of hydraulic grade line (land slope, ft/ft).

Shallow Concentrated Flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from the following figure in which average velocity is a function of watercourse slope and type of channel (TR-55). The flow is still considered shallow in depth and flows in a swale or gutter instead of a channel, which has greater depth.

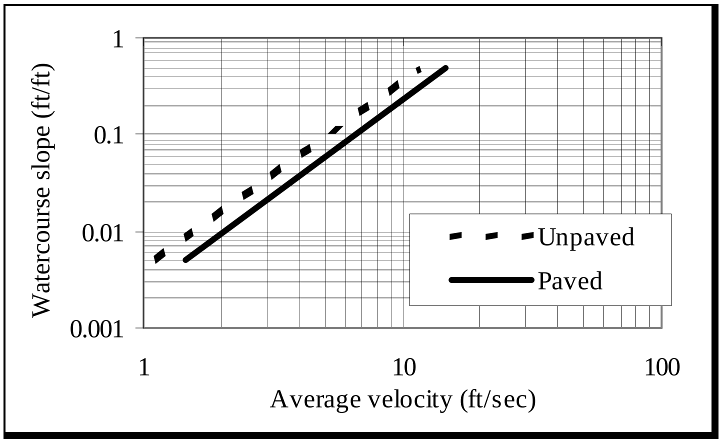


Figure 7. Avg. Velocities for Estimating Travel Time in Shallow Concentrated Flow Segments

After determining the average velocity, the following equation is used to compute travel time:

$$T_T = L / (3600 * V) \qquad \text{Equation 9}$$

Where:

T_T	=	travel time (hr),
L	=	flow length (ft),
V	=	average velocity (ft/sec), and
3,600	=	conversion factor from seconds to hours.

Channelized Flow

As the depth of concentrated flow increases, the shallow concentrated flow evolves into channelized flow. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle maps. In the case of this analysis, channel flow either involves flow in man-made storm sewer infrastructure or flow in the natural channel. Manning's equation or water surface profile information (available from HEC-2 or HEC-RAS) can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevations. Both open channel and closed conduit systems can be included.

Manning's equation is:

$$V = 1.49 * r^{2/3} * s^{0.5} / n \qquad \text{Equation 10}$$

Where:

V	=	average velocity (ft/sec),
r	=	hydraulic radius (ft), equal to flow area divided by wetted perimeter,
s	=	slope of the hydraulic grade line (channel slope, ft/ft), and
n	=	Manning's roughness coefficient.

3.5.2 Kerby-Kirpich Method

The Kirby-Kirpich method estimates the time of concentration by calculating and summing the travel time of two components of flow: overland flow and channel-flow. The Time-Parameter Estimation for Applicable Texas Watersheds (TxDOT, 2005) report supports the use of the Kerby-Kirpich method for estimating time of concentration for Texas watersheds. Research concluded that times of concentration estimated with the Kirpich method were less variable than estimates made with the NRCS travel-time method. The Kirpich method was also easier to use and repeat than the NRCS method due the smaller number of parameters. Input parameters for the Kirpich method are more consistently applied, as these parameters are available from published resources, whereas the selection of NRCS parameters relies heavily on engineering judgment. Also, research showed the time to peak estimated with the Kerby-Kirpich method is consistent with actual storm hydrographs. Time of concentration calculations with the Kerby-Kirpich method are included in Appendix D.

Overland Flow

The Kerby method is applicable for calculating the overland flow time for small watersheds where overland flow is an important component of overall travel time. The flow is considered shallow in depth and flows in a swale or gutter instead of a channel, which has greater depth.

The following equation is used to compute overland flow travel time:

$$Tc = K(L*N)^{0.467} S^{-0.235} \qquad \text{Equation 11}$$

Where:

Tc	=	overland flow time of concentration (min),
K	=	units conversion coefficient, K = 0.828
L	=	overland flow length (ft),
N	=	dimensionless retardance coefficient
S	=	dimensionless slope of terrain conveying the overland flow

Values of the retardance coefficient range from 0.1 for bare and packed soil to 0.8 dense grass or forest. A retardance coefficient of 0.1 was applied to fully developed subbasins. A retardance coefficient of 0.3 was applied to subbasins that are not fully developed. A maximum length of 1,000 feet was used as a maximum overland flow length.

Channelized Flow

As the depth of flow increases, the overland flow evolves into channelized flow. In the case of this analysis, channel flow either involves flow in man-made drainage ditches or flow in the natural channel. The Kirpich equation was used to estimate the channel-flow component of time of concentration.

Kirpich equation is:

$$Tc = K*L^{0.770}S^{-0.385} \qquad \text{Equation 12}$$

Where:

Tc	=	time of concentration (min),
K	=	units conversion coefficient, K = 0.0078
L	=	channel flow length (ft),
S	=	dimensionless main-channel slope

3.6 HYDROGRAPH ROUTING

Channel routing simulates the movement of a flood wave through a reach, allowing for the prediction of variation in time and space. Hydrologic routing allows runoff hydrographs from multiple subbasins to be combined and routed to a point of interest. The following sections describe two methods used to route: modified Puls and Muskingum-Cunge methods. These methods are considered hydrologic methods of routing, which uses the continuity equation and a relationship between reach storage and discharge at the outlet. Further description of the routing method with respect to model calibration is described in Section 4.0.

3.6.1 Modified Puls Method

The modified Puls method is a routing technique that relates storage, outflow, and water surface slope in a river reach. In a natural river, storage is a function of outflow and a function of water surface elevation. To define a unique storage-discharge relationship, the channel is broken into several segments, or steps, with each segment treated as a level pool reservoir.

The number of routing steps is defined as the wave travel time divided by the time step (HMS computation interval). Travel time is defined as the reach length divided by average wave celerity. Wave celerity can be estimated as the slope of the discharge rating curve divided by the top width of the water surface. As a rule of thumb this value of celerity can be approximated by multiplying 1.5 times the average flow velocity, for natural channels.

As the number of time steps for a routing reach increases, the flood attenuation for that reach decreases. Typically, the number of steps is selected such that the travel time through the reach is approximately equal to the time step. As a result, reaches with a low velocity have a relatively large number of steps. The approach used by Tropical Storm Allison Recovery Project (TSARP) in *Recommendation for: Routing Steps with HEC-HMS* is to consider a reach as functioning as linear reservoir with a time step of 1, if the average velocity is less than 1.0 feet per second. This model assumes that reaches with velocities less than 0.5 feet per second are considered linear reservoirs.

The modified Puls routing method accounts for channel and overbank storage. In flat areas, such as the Texas coast, channel and overbank storage has a significant influence on watershed hydrology. The modified Puls method was used on North Main Drain and its tributaries.

3.6.2 Muskingum-Cunge Method

The Muskingum-Cunge method is used to modify hydrographs to reflect the effects of translation and attenuation within a channel reach. The input parameters of this method are physical channel properties: channel length, channel slope, Manning's roughness coefficients, and the friction slope or channel bed slope. A trapezoidal channel shape is used to represent a typical channel section through each stream routing reach. Storage is a function of inflow and outflow, weighted by travel time through the reach. Hydrograph diffusion is based on channel properties and the inflowing hydrograph. With the Muskingum-Cunge method, hydrographs can be determined at individual cross sections. This method yields less wave attenuation, permitting flexibility in choice of time and space increment, but does not take into account backwater effects (Chow et al, 1988). Muskingum-Cunge was used for Stuart Place Main, Dixieland Main and Southwest Main. In these watersheds flow is generally channelized and overbank storage is less of an issue.

3.7 DESIGN STORM ANALYSIS

The application of a design storm in the HEC-HMS model is used to generate runoff hydrographs and estimate peak flow rates along the watercourse for various storm frequencies. There are three major components to the design storm: depth, duration, and distribution. Precipitation depths that have been selected for this impact study are included in Section 2.2. The following subsections describe the analysis and selection of storm duration and distribution.

3.7.1 Design Storm Duration

Design storm duration is a significant consideration for hydrologic modeling. The peak flow of any given event must reach the mouth of the studied basin prior to the end of the rainfall duration. A 48-hour design storm was selected for this analysis. This design storm duration exceeds the largest time of concentration of the drainage areas.

3.7.2 Design Storm Distribution

A balanced and nested distribution is assumed for this analysis due to its flexibility with regard to storm duration. The distribution is balanced in that the precipitation is centered about the center of the duration. The distribution is nested in that the precipitation depths from the USGS publication are applied in an alternating block format (i.e., the 15-minute depth is applied as the hyetograph peak, the 30-minute depth is applied such that the peak 15-minute block and the adjacent 15-minute block sum to be the 30-minute depth).

3.8 ULTIMATE CONDITIONS ANALYSIS

The ultimate development conditions (fully developed conditions) analysis uses the validated existing conditions basin model and the balanced and nested distribution to determine the flow rates for the watersheds at full development. For the purposes of this analysis, full development is equivalent to a 30-year time horizon (i.e., the development status in the year 2037).

The time of concentration was adjusted to reflect shorter watershed response times, specifically in the uplands of the watershed, through the Kerby equation's retardance coefficient. The existing conditions use a retardance coefficient of 0.3 for the majority of the subbasins. The retardance coefficient was lowered to 0.1 for ultimate conditions.

This ultimate watershed conditions analysis includes flow rates for the 1% annual chance (100-year) only. These ultimate conditions flow rates are used to determine the ultimate conditions floodplain for the 1% annual chance event.

4.0 HYDRAULIC ANALYSIS

The hydraulic analysis is conducted on reaches within four major drainage ditch networks in the CCDD5 watershed. There are 51 miles of stream included with this hydraulic analysis, which computes water surface elevations for the 50%, 10%, 4%, 1%, and ultimate 1% annual chance (2-, 10-, 25-, 100-, and ultimate 100-year, respectively) storm events. The hydraulic analysis includes the delineation of the 1% annual chance as well as the ultimate 1% annual chance floodplains. The studied drainage networks include the following:

- North Main Drain
- Stuart Place Main Drain
- Dixieland Main Drain, and
- Southwest Main Drain.

An overall drainage map showing the extents of the studied reaches is included as Exhibit 1 in Appendix A of this report. In total, there are 39 hydraulic reaches that include 67 modeled structures. The specific studied reaches are included as Exhibits 5 through 8 in Appendix A.

The hydraulic analysis performed in this study does not assume any backwater effects from the Arroyo Colorado, as peak flows of the Arroyo Colorado and the drainage networks are not expected to coincide. This assumption is consistent with the 1990 USACE Feasibility Study of Cameron County.

The USACE HEC-RAS software version 3.1.3 is used for the hydraulic analyses. All modeling is one dimensional and steady state. The sections that follow describe the development of the hydraulic models both in general terms and specifics that apply to certain reaches.

4.1 DESCRIPTION OF HYDRAULIC MODEL GENERATION

Separate HEC-RAS models were generated for the four subwatersheds: North Main, Stuart Place Main, Dixieland Main, and Southwest Main. The hydraulic models were generated using 2005 LIDAR contour data, field-surveyed cross sections, and field-surveyed structures. Each of these networks consists of man-made channels with mostly grass bottom and grass side slopes. Stream centerlines and cross sections were created with ArcMap and imported into HEC-RAS using Geo-RAS software. All cross sections are modeled from left to right looking downstream.

All networks were modeled under a subcritical flow regime, which is consistent with FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners*, Appendix C.3.4.4. Downstream boundary conditions were assumed to be normal depth with a slope of 0.4 percent. The table below lists the stream length and number of reaches, cross sections, culverts and bridges modeled for each network in HEC-RAS.

Table 7. CCDD5 HEC-RAS Models

HEC-RAS Model	Stream Length (mi)	Quantitity			
		Reaches	Cross Sections	Culverts	Bridges
North Main	37.1	30	420	30	20
Stuart Place	10.5	7	125	7	4
Dixieland	2.3	1	48	6	0
Southwest	1.2	1	22	0	0
Total	51.1	39	615	43	24

The North Main is the largest model with more reaches, cross sections, surveyed culverts and bridges than the other three networks combined.

4.2 HYDRAULIC MODEL DEVELOPMENT

4.2.1 Streamlines and Cross Section

Study streamlines and cross sections are created using ArcGIS 9.2 and LIDAR. Cross sections along the streamlines were placed to capture natural cross sections and data for hydraulically significant structures, including bridges, culverts, and roads. A map of cross section location for each model is included in Appendix A as Exhibits 5 through 8. An extensive field survey of important hydraulic structures was conducted to help enhance the accuracy of the hydraulic model. This data was imported into HEC-RAS software using HEC-GeoRAS tools. Survey data sheets are included as a PDF file in Appendix J.

4.2.2 Parameter Estimation

Hydraulic models require several estimated parameters, including the Manning's roughness coefficients for channels and overbanks, contraction and expansion coefficients, and ineffective limits.

Manning roughness coefficient, n, is a measure of the roughness of channels and overbanks. The value n varies with flow depth, alignment, amount and type of vegetation, and flow obstructions. The table below lists typical values for Manning's n. For all hydraulic models in the CCDD5 FPP use a Manning's roughness coefficient of 0.045 for channels, and 0.08 for overbanks.

Table 8. Manning Roughness coefficients for various open channel surfaces

Material	Typical Manning roughness coefficient
Concrete	0.012
Gravel bottom with sides	
concrete	0.020
mortared stone	0.023
riprap	0.033
Natural stream channels	
Clean, straight stream	0.030
Clean, winding stream	0.040
Winding with weeds and pools	0.050
With heavy brush and timber	0.100
Flood Plains	
Pasture	0.035
Field crops	0.045
Light brush and weeds	0.050
Dense brush	0.070
Dense trees	0.100

Source: Chow, et al. 1988

Contraction and expansion coefficients are applied upstream and downstream, respectively, of culverts and bridges to model the contraction and expansion of flow. In this study, contraction and expansion coefficients of cross sections bounding bridges and culverts is 0.3 and 0.5, respectively. All other cross sections use the default contraction and expansion coefficients of 0.1 and 0.3.

Table 9. Miscellaneous Hydraulic Coefficients Table

Coefficient Type	Value or Range
Bridge pier drag coefficient for momentum equation applications, Cd	2
Pressure and weir flow coefficient (submerged inlet and outlet), Cd	0.8
Expansion coefficients for bridges / culverts / in-line structures	0.3 to 0.5
Expansion coefficients for channels	0.3
Contraction coefficients for bridges / culverts / in-line structures	0.1 to 0.3
Contraction coefficients for channels	0.1
Weir coefficients (road deck)	2.6 to 3.0
Culvert entrance loss coefficient	0.4
Culvert exit loss coefficient	1

Ineffective flow limits are added to cross sections to accurately model any given section's inability to convey flow, such as cross sections that bound bridges and culverts. Ineffective limits were also set at the top of the channel banks to account for storage in overbanks that do not contribute to channel conveyance. Storage must be accounted to accurately model with modified Puls routing.

5.0 HYDROLOGIC AND HYDRAULIC MODEL VALIDATION

5.1 HYDROLOGIC VALIDATION

5.1.1 Background

Limited historical flow data is available for CCCD5. On May 25, 2007, a storm in the CCDD5 area occurred which provides the most recent and comprehensive data for this area; therefore, the validation exercise focuses on this event. The data collected for the May 25th event included measured high water marks taken along North Main drainage network recorded by Alan Moore, Drainage District #5 General Manager, and 15-minute precipitation depths recorded at three rain gages located at various pump stations. The measured high water marks are listed in Table 10.

In the northeast portion of CCDD5, the precipitation was similar to the 25-year event. However, in the southeast portion of CCDD5 did not receive the same intensity and the storm was close to a 2-year event. The runoff for this event was approximately a 10-year event. Composite hyetographs were computed for each of the 59 subbasins using the inverse square method to account for the non-uniformity of the storm intensity. Rainfall data of the May 25, 2007 event is included in Appendix E.

Table 10. May 25th, 2007 Event, CCDD5

Location	WSEL (ft)
Chester Park Road	36.2
Wilson Road	36.03
Expressway 77	35.75
Business 77	34.82
Loop 499	34.27
New Combes Hwy	33.81
Breedlove Street	34
Briggs Road	33
FM 499	31
FM 507	29
FM 1420	11

5.1.2 Calibration

The hydrologic model for the North Main watershed was calibrated to simulate the May 25th event observed water surface elevations. The table below lists the six North Main hydrologic models created for calibration with the May 25th event.

Table 11. Calibration models for May 25th Event

	Calibration Number	Antecedent Moisture Condition	Time of Concentration	Channel Routing	Peak Rate Factor
	1	AMC I	TR-55	Muskingum-Cunge	484
Standard	**2**	**AMC II**	**TR-55**	**Muskingum-Cunge**	**484**
	3	AMC I	Kerby-Kirpich	Muskingum-Cunge	484
	4	AMC II	Kerby-Kirpich	Muskingum-Cunge	484
Selected	**5**	**AMC I**	**Kerby-Kirpich**	**Modified Puls**	**200**
	6	AMC II	Kerby-Kirpich	Modified Puls	150

Parameters analyzed for calibration are the antecedent moisture conditions, time of concentration calculation method, channel routing method, and the peak rate factor of the unit hydrograph. Calibration number 5 was selected as the most appropriate. Parameters of the selected model are discussed below.

- **Antecedent Moisture Conditions**

Standard methodology uses curve numbers based on AMC II, which is valid for average soil conditions in the United States. The selected model uses AMC I, as it is more appropriate for the dry soil conditions with low runoff potential found in Cameron County.

- **Peak Rate Factor**

Standard methodology uses a peak rate factor of 484 for the user-specified unit hydrograph. The flat terrain in this study area requires a lower peak rate factor. A peak factor rate of 150 was used in calibration number 6, but the selected calibration uses 200, as it yielded a water surface elevations for the May 25th event closer to those observed during the storm. A peak rate factor of 200 was applied to the entire CCDD5 watershed.

- **Time of Concentration**

Standard methodology uses the TR-55 method to calculate time of concentration. The selected model uses Kerby-Kirpich method as it was found to be more reliable for Texas watersheds and used parameters that were less subjective. The Kirby-Kirpich method was used to calculate time of concentrations for all the basins in the CCDD5 watershed.

- **Channel Routing**

The modified Puls routing was determined to be more appropriate to use over the North Main network. In the North Main watershed out-of-channel flow is common, making storage in the overbanks a factor to be considered and accounted for in hydraulic modeling. Muskingum-Cunge routing was applied to the drainage ditches in the Stuart Place, Dixieland and Southwest watersheds. Stuart Place Main is generally contained in the channels, making out-of-bank storage less significant. Storage is not significant in Dixieland and Southwest watershed, as these networks are relatively short.

The figure below shows the water surface elevation under the standard and selected hydrologic models for the May 2007 rainfall. The standard model's water surface elevation is labeled as WS May25_SCS_AMC2. The selected model's water surface elevation is labeled as WS May 25.

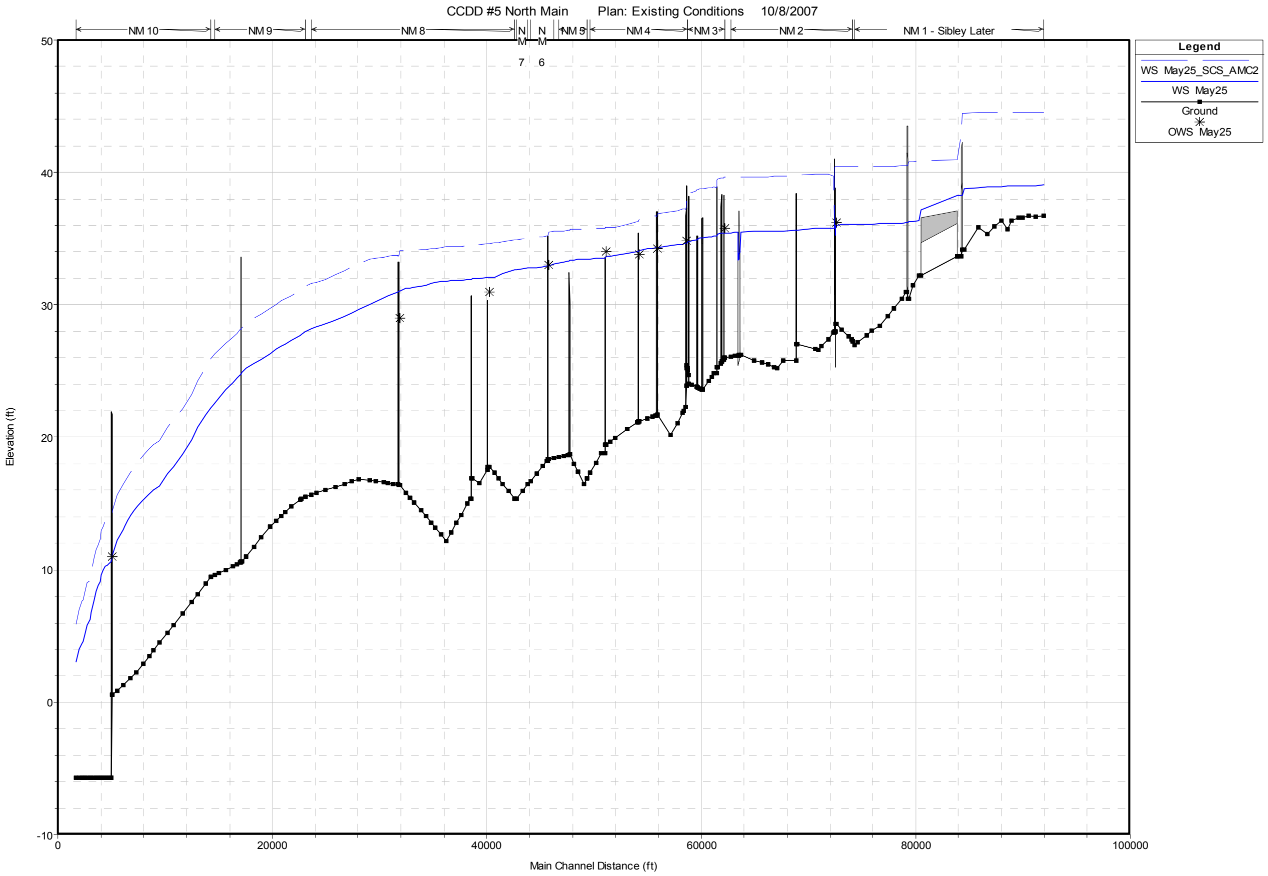


Figure 8. Comparison of Standard vs. Selected Hydrologic Models

The selected model's calculated water surface elevation lies closer to the observed water mark elevations than the standard methodologies. The parameters that best calibrate with the May 25th event are AMC I, Kerby-Kirpich method for time of concentration, and a PRF of 200. These parameters were used throughout the CCDD5 watershed. Modified Puls routing is applied to the drainage network on the North Main watershed, and Muskingum-Cunge routing is applied to those in the Stuart Place, Dixieland and Southwest watersheds.

5.2 COMPARISON WITH PREVIOUS STUDIES

The objective of the 1990 USACE Feasibility Report for Cameron County, Texas report was to determine the feasibility of Federal participation in flood control measures to reduce flood damages. In this study, HEC-1 and HEC-2 models were developed for several channels in Cameron County. HEC-1 models were used to determine peak flow rates. HEC-2 models were used to determine corresponding peak stage. A description of the model created and a comparison with the CCDD5 FPP models are discussed in the following sections.

5.2.1 Comparison of Hydrologic Data

Of the channels selected for the USACE Feasibility study only portions of the present North Main network are included. Under the feasibility study, these man-made channels are classified as tributaries in urban basins. Rainfall depths are taken from TP-40 and TP-49. The event duration of 96 hours with a computation interval varying from 15 minutes to 1 hour is used. The Standard Project flood used rainfall depths of 50 percent of probable maximum rainfall as taken from Hydrometerological Report No. 51 (HMR 51) published by the National Weather Service. An average SCS curve number of 80 was originally used to estimate rainfall losses, but modified as calibration revealed that a curve number of 65 was more reasonable. Modified Puls routing method was used. Peak flow rates for the 100-year, no-project channel design in the feasibility study are listed in the table below for comparison with CCDD5 FPP 100-year existing peak flow discharges.

Table 12. Peak Flow Rate Comparison of 1990 USACE Feasibility Study and CCDD5 FPP

	USACE				Espey		
Estimated Location	Node	Reach	Station	100-yr Discharge (cfs)	Reach	Station	100-yr Discharge (cfs)
Near outfall to Arroyo Colorado	1	AN-13	1309	3020	NM 10	1701	3300
Before confluence with McLeodHood Reservoir Lateral	2	AN-13	13340	2278	NM 10	14237	3023
After confluence with NMT 9 (508 Crossing Lateral)	3	AN-13	20735	2123	NM 9	23092	3013
Before confluence with NMT 9 (508 Crossing Lateral)	3	AN-13	22420	1685	NM 8	23635	2823
3000 ft upstream of FM 507	4	AN-13	32281	1692	NM 8	34785	2823
After confluence with NMT 7 (13th St Lateral)	5	AN-13	41736	2333	NM 7	43830	2435
Before confluence with NMT 7 (13th St Lateral)	5	AN-13	43477	1731	NM 6	44082	2125
After confluence with NMT 5 (Zavala Lateral)	6	AN-13	43412	2243	NM 5	49385	2002
Before confluence with NMT 5 (Zavala Lateral)	6	AN-13	48720	1289	NM 4	49654	1688
After confluence with NMT 43 (Primera-Hand Rd Lateral)	7	AN-13	56447	1423	NM 4	58653	1564
Before confluence with NMT 43 (Primera-Hand Rd Lateral)	7	AN-13	56497	741	NM 3	58767	1156
Chester Park Rd.(Hand Rd.)	8	AN-13	68891	669	NM 2	72352	636
Before confluence with Bus 77	7	AN 13-07	0	708	NMT 43 R 3	71	496
Hand Rd.	10	AN 13-07	8250	965	NMT 43 R 3	8446.1	519
Wilcox Rd.	11	AN 13-07	14200	717	NMT 42 R 1	4068	296

5.2.2 Comparison of Hydraulic Data

The USACE Feasibility Study only presents the water surface elevations for the standard project flood, which is defined as 50 percent of probable maximum rainfall as taken from HMR 51. The standard project flow was not studied in this analysis the CCDD5 FPP.

The use of a normal depth boundary condition in the CCDD5 FPP is consistent with modeling in the USACE Feasibility Study. The USACE feasibility uses a critical depth boundary condition, but backwater computations do not affect the upstream reaches because of the steep gradient the first thousand feet of channel. Both models assume non-coincident peaks between the Arroyo Colorado and the drainage ditches (USACE, 1990).

5.3 HYDROLOGIC ANALYSIS SUMMARY AND CONCLUSIONS

This hydrologic analysis evaluates the CCDD5 watershed. Based on the results of this analysis, the most appropriate design storm for this study is the balanced and nested distribution with a 48-hour duration. The nested USGS precipitation depths are applied to this distribution. The most appropriate basin model for this analysis is the validated model discussed earlier in the text. For ultimate watershed conditions, the existing conditions basin model is revised to reflect projected future impervious cover based on the composite future land use map. Results of this hydrologic analysis will be used to delineate the floodplains discussed later in this report.

6.0 FLOODING ANALYSIS OF CCDD5

6.1 EXISTING CONDITIONS FLOODING ANALYSIS

The hydrologic and hydraulic models were used to evaluate the existing conditions of the North Main Drain, Stuart Place Main, Dixieland Main, and Southwest Main drainage ditches. Water surface elevations for the 2-, 5-, 10-, 25- and 100-year storm events under existing conditions (current land use), calculated with HEC-RAS, were imported into GIS software (ArcMap 9.2) to identify out-of-bank flooding and water depths. The results of the existing conditions analysis are summarized in the following sections in terms of level of service provided by the ditches. The level of service of each reach is a measure of the magnitude of the storm event, in terms of frequency, that the ditch can contain without out-of-channel flooding. The existing conditions floodplain maps for each watershed for the 10% and 1% storm events are included in Appendix A as Exhibits 9-12. The floodplain maps do not represent the entire floodplain, just the calculated flooding limits to the limits of the modeled cross-sections. The floodplain is not contained by high ground in many locations, and apparent limits of the floodplain show only the limits of the hydraulic model and not the full extent of flooding.

6.1.1 North Main

The level of service of the North Main Drain varies along the network. Several areas have a low hydraulic capacity, and therefore are more prone to flooding. Along the main stem, reaches extending from Dilworth Road to New Combes Highway and from Breedlove Street to just downstream of FM 507 provide less than a 2-year level of service. The laterals draining the Town of Primera also have a low hydraulic capacity with only 2-year level of service

Areas providing the highest level of capacity include reaches extending from Searcy Ranch Road to the Arroyo Colorado along the main stem and from US 77 to Crossett Road along Wilson Tract Main lateral. Table 13 summarizes the level of service of the North Main Drain drainage ditches.

Table 13. Existing Level of Service of North Main Drain

Reach	Level of Service
North Main Drain Main Stem	
Top of reach to Dilworth Road	5-year
Dilworth Road to New Combes Highway	2-year
New Combes Highway to Breedlove Street	10-year
Breedlove Street to Briggs Coleman Road	2-year
Briggs Coleman Roadto Heoning Road	less than 2-year
Heogning Road to approximately 2000' downstream of FM 507	2-year
Approximately 2000' downstream of FM 507 to Searcy Ranch Road	5-year
Search Ranch Road to Arroyo Colorado	25-year
13 St Lateral	
499 North to Flores Drive	5-year
Flores Drive to North Main main stem	2-year
Zavalata Lateral	
499 North to Montezuma Road	10-year
Montezuma Road to North Main main stem	2-year
David Stephen Lateral	
Mont Park Drive to Breedlove Street	5-year
Breedlove Street to North Main main stem	2-year
All States Lateral	
Montana Drive to Iowa Drive	less than 2-year
Iowa Drive to North Main main stem	10-year
Wilson Tract Main Lateral	
US 77 to Crossett Road	25-year
Crossett Road to Breedlove Street	10-year
Breedlove Street to Briggs Coleman Road	5-year
Briggs Coleman Road to North Main main stem	2-year
508 Crossing Lateral	
Upstream of FM 508 to North Main main stem	5-year
McLeod Hood Lateral	
FM 508 to McCloud Road	10-year
McCloud Road to North Main main stem	25-year
Las Palmas Bypass Lateral	
FM 2994 to McCullough Street	10-year
McCullough Street to North Main main stem	2-year
Carters Lateral	
Hands Road to North Main main Stem	2-year
Wilson Tract Main Lateral	
Tamm Lane to Young Lateral	2-year
Murphy Primera Lateral	
Nacahuita to Primera Lateral	2-year
South Fork Lateral	
FM 2994 to Primera Road	2-year
Primera Lateral	
Murphy Primera Lateral to Primera Hand Lateral	2-year
Primera Hand Lateral	
Young Lateral to North Main main stem	2-year

6.1.2 Stuart Place Main

The hydraulic capacity of Stuart Place Main drainage ditches varies along the network. The main stem provides a 100-year level of service but the laterals pose a more significant risk to flooding. The Old Stuart Place lateral provides a 25-year level of service from the top of the reach to FM 3915, and a 100-year level of service from FM 3915 to the confluence with Stuart Place main stem. The Hensz lateral provides a 10-year level of service. The Payne lateral provides a 10-year level of service from the top of the reach to Palm Boulevard, and 25-year level of service from Palm Boulevard to the confluence with Stuart Place main stem.

Table 14. Existing Level of Service of Stuart Place Main

Reach	Level of Service
Stuart Place Main Drain Main Stem	
Bougainvillea Drive to Arroyo Colorado	100-year
Old Stuart Place Lateral	
Top of reach to FM 3915	25-year
FM 3915 to Stuart Place main stem	100-year
Hensz Lateral	
Doan Road to Payne Lateral	10-year
Payne Lateral	
Top of reach to Palm Boulevard	10-year
Palm Boulevard to Stuart Place main stem	25-year

6.1.3 Dixieland Main

Dixieland Main drainage ditch upstream of Garrett Street poses a significant risk to flooding. The culvert at Bothwell Street obstructs flow at less than a 2-year event. This obstruction results in overbank flooding as far downstream as Garrett Street. Dixieland Main downstream of Garrett Street provides a high hydraulic capacity as shown in the table below.

Table 15. Existing Level of Service of Dixieland Main

Reach	Level of Service
Dixieland Main Drain Main Stem	
Lincoln Road to Bothwell Street	less than 2-year
Bothwell Street to Garrett Street	2-year
Garrett Street to Dixieland Street	25-year
Dixieland Street to Arroyo Colorado	100-year

6.1.4 Southwest Main

The Southwest Main provides a low hydraulic capacity upstream of Cook Lane. The overbanks sit at a lower elevation upstream of Cook Lane, resulting in out-of-channel flows for more frequent events.

Table 16. Existing Level of Service of Southwest Main

Reach	Level of Service
Southwest Main Drain Main Stem	
Atlas Palmas to 1000 ft downstream	5-year
1000 ft downstream of Atlas Palmas to Cook Lane	10-year
Cook Lane to Arroyo Colorado	100-year

6.2 ULTIMATE CONDITIONS FLOODING ANALYSIS

The hydrologic and hydraulic models were used to evaluate the ultimate conditions of the North Main Drain, Stuart Place, Dixieland and Southwest ditches. Ultimate conditions imply future land uses with existing structures. These models are used to identify out-of-bank flooding and water depths. These findings are summarized in the following sections.

6.2.1 North Main

In general, the 25-year ultimate water surface elevations are comparable to the 100-year existing water surface elevations along the North Main Drain main stem and laterals. The table below compares the 10-year and 100-year existing and ultimate conditions discharges at various locations along the North Main Drain main stem.

Table 17. Percent Change for 10-yr and 100-yr Peak Flow Rates under Existing and Ultimate Conditions

10-Year Peak Flow Rate (cfs)			100-Year Peak Flow Rate (cfs)			
Existing	Ultimate	Percent Change (%)	Existing	Ultimate	Percent Change (%)	Location along North Main Drain main stem
17	70	320	71	153	116	FM 2994
169	323	91	405	577	43	Dilworth Road
237	380	60	461	634	37	Chester Park Road
731	1037	42	1493	1834	23	Industrial Way
885	1236	40	1840	2191	19	Business 77
1024	1525	49	2048	2755	35	Breedlove Street
1258	1760	40	2348	3121	33	U/S of confluence with David Stephen Lateral
1593	2224	40	3025	4001	32	D/S of confluence with Wilson Tract Main
1613	2283	42	3107	4092	32	FM 508
1546	2326	50	3061	4094	34	FM 507
1593	2520	58	3226	4330	34	Searcy Ranch Road
1689	2684	59	3317	4633	40	Outfall to Arroyo Colorado

6.2.2 Stuart Place Main

Along the Stuart Place main stem, the 10-year ultimate event water surface elevation is comparable to the 100-year existing. Along Old Stuart Place lateral, the 100-year existing water surface elevation falls between the 2-year and 5-year ultimate water surface elevations. The 100-year existing water surface elevation along the Hensz and Payne laterals falls between the 10-year and 25-year ultimate water surface elevations.

The hydraulic capacity of Stuart Place drainage ditches is reduced under ultimate conditions. The Stuart Place main stem hydraulic capacity is lowered to from a100-year to a 25-year level of service from the top of the reach to downstream of Garrett Road. The hydraulic capacity of Old Stuart Place Main is reduced from a 25-year to a 2-year level of service from the top of the reach to 1,250 ft upstream of FM 3195. The hydraulic capacity of the Hensz lateral reduces form a 10-year level of service to less than a 2-year level of service. The hydraulic capacity along Payne lateral from the top of the reach to 1,000 ft downstream of Palm Boulevard reduces from a 10-year level of service to a 2-year level of service.

Table 18. Percent Change for 10-yr and 100-yr Peak Flow Rates under Existing and Ultimate Conditions

10-Year Peak Flow Rate (cfs)			100-year Peak Flow Rate (cfs)			
Existing	Ultimate	Percent Change (%)	Existing	Ultimate	Percent Change (%)	Location along Stuart Place Main Drain main stem
26	60	126	76	119	57	Stuart Place Road
171	548	220	451	1022	126	US Hwy 83
234	774	231	609	1464	141	Railroad
311	938	202	804	1780	121	Dilworth Road
338	991	193	892	1887	111	Garrett Road
507	1409	178	1417	2741	93	South Palm Court Drive

6.2.3 Dixieland Main

Along Dixieland Main drainage ditch the 100-year existing water surface elevation is comparable to the 25-year ultimate water surface elevation, the 25-year existing to 5-year ultimate and 10-year existing to 2-year ultimate.

The hydraulic capacity of Dixieland Main drainage ditches is reduced under ultimate conditions. The hydraulic capacity between Garrett Street and Dixieland Street reduced from a 25-year level of service to a 5-year level of service. The hydraulic capacity along the channel 500 ft downstream of Dixieland reduces from a 100-year level of service to a 25-year level of service.

Table 19. Percent Change for 10-yr and 100-yr Peak Flow Rates under Existing and Ultimate Conditions

10-Year Peak Flow Rate (cfs)			100-Year Peak Flow Rates (cfs)			
Existing	Ultimate	Percent Change (%)	Existing	Ultimate	Percent Change (%)	Location along Dixieland Main
52	108	107	137	215	57	Bothwell Road
113	230	103	294	454	54	Dixieland Street
206	436	111	540	868	61	Outfall to Arroyo Colorado

6.2.4 Southwest Main

Along the Southwest drainage ditch, the 100-year existing water surface elevation lies between the 10-year and 25-year ultimate water surface elevation. The 100-yr existing water surface elevation is between 0.10 to 0.20 feet lower than the 25-year ultimate water surface elevation along the channel.

Under ultimate conditions, the hydraulic capacity from Atlas Palmas to Doan Road reduces from a 5- to 10-year level of service to a less than 2- to 2-year level of service. The capacity from Cook Lane to the Arroyo Colorado remains at a 100-year level of service.

Table 20. Percent Change for 10-yr and 100-yr Peak Flow Rates under Existing and Ultimate Conditions

10-Year Peak Flow Rate (cfs)			100-Year Peak Flow Rate (cfs)			
Existing	Ultimate	Percent Change (%)	Existing	Ultimate	Percent Change (%)	Location along Southwest Main
59	151	157	178	304	71	S. Altas Palmas Road
74	187	151	230	376	63	Cook Lane

7.0 ALTERNATIVES ANALYSIS

7.1 PLANNING A SYSTEM OF IMPROVEMENTS

The purpose of the Flood Protection Plan is to evaluate the relative benefits of the mitigation strategies developed herein, in order to guide the District in selecting, prioritizing and implementing an optimized combination of strategies. Costs presented herein are for comparison of potential capital improvement projects. To assist CCDD5 in prioritizing which projects should be funded, the alternatives are assessed with a combination of cost of implementation and associated benefits. Evaluated projects include structural flood controls and non-structural measures.

Structural flood controls are potential construction projects that could be built in an effort to alter the flooding condition of a watershed. Examples of structural controls include culvert improvements, channel maintenance, construction of detention ponds, and diversions. Structural controls mitigate flooding by rerouting, detaining, or altering the hydraulics of flow. These controls typically incur significant construction expenses, and cost associated with right-of-way acquisition. Structural improvements that increase conveyance capacity (increased channel/culvert size) will typically reduce the amount of storage in the system by reducing ponding and overbank flooding. Changes to system storage must be carefully analyzed for this area since reductions in storage can reduce the amount of peak flood attenuation and increase flow rates downstream of improvements. The impacts of structural improvements need to be assessed for the entire system downstream of the improvement to insure that no additional damage is caused as a result. For this reason it is expected that some structural improvements considered will result in only localized benefits and result with significant damages downstream. This will be further discussed below in the consideration of each respective alternative.

Non-structural flood control measures, typically in the form of community-based initiatives and programs, may prevent the worsening of flood problems and aim to prevent flood-induced hazards. Examples of non-structural flood control measures include flood alert systems and buy-outs in flood prone areas. Non-structural controls aim to control the land use of flood-prone areas and to restrict timing and reduce runoff. As much of the success of non-structural measures is found when implemented during the course of new development, system-wide runoff control/impact fee policies for new development will likely be an important component for CCDD5, given the amount of undeveloped area in its jurisdiction.

Implementing both types of controls typically provides the best results. Structural controls are designed to optimize conveyance of peak flows. Non-structural controls often prevent an increase in runoff, maintaining the peak discharge, so that structural controls will continue to be effective; or, these controls seek solutions to other dimensions of the flooding problem, such as awareness and response.

7.2 COST BENEFIT ANALYSIS

The viability of alternatives is primarily measured through a comparison of the relative costs and benefits. The estimated costs for each alternative include materials and construction costs, which are based on recent bid tabulations and unit prices for similar regional construction projects. Several of the assumptions used to estimate costs also include:

- Excavation: $2 / CY (reflecting the known in-house cost to the District for excavation)
- 10' by 10' Box Culvert: $800 / LF (reflecting the cost known to the District from recent bids)
- Mobilization: 5 %
- Engineering & Surveying: 20 %
- Contingency: 20 %.

It was not within the scope of this study to perform more detailed net present value analysis or other time-weighted analysis.

The benefit of the alternative is the relative monetary savings of a given improvement being in-place, compared to it "not being in-place". This value is determined from the difference between estimated damages for existing condition and estimated damage with alternative in-place for the 10-year and 100-year events.

7.2.1 Methodology of Estimating Benefits

Flatland conditions generate different flood damage patterns than steep-gradient "flash flood" terrain. In flat areas, damage is primarily a function of depth and duration of inundation. With this in mind, the algorithm developed for estimating flood damage in CCDD5 is based upon varying depth.

The benefit of the alternative is the relative monetary savings of a given improvement being in-place, compared to it "not being in-place". This value is determined from the difference between estimated damages for existing condition and estimated damage with alternative in-place for the 10-year and 100-year events. The 25-year event selected as the desired level of service that the District would like to achieve with their drainage system, however, the 10-year and 100-year events were chosen for analysis because these two clearly demonstrate the extent of flooding problems in these areas. The 100-year event is also analyzed, as this is the primary return interval used by the NFIP.

To estimate the risk associated with a given magnitude flood event, HAZUS-MH software was employed. This software, developed by FEMA Hazard Mitigation Division under a contract with the National Institute of Building Sciences, integrates with ArcGIS 9.2 (the platform utilized for spatial data management and analysis in the overall study). HAZUS is increasingly a widely-accepted methodology for flood damage estimation. HAZUS provides an estimate of damages by taking spatial information about the depth of flooding, and correlating that information in an "overlay" analysis to data about the built environment and regional assumptions about the relationship between depth of inundation and damages. In addition to this information, HAZUS provides other useful emergency management data such as estimates of displaced households, disrupted critical facilities, and business use loss.

For the District's purposes, HAZUS was used to generate estimates of the relative benefit of the flood protection measures proposed. The results of the hydraulic analysis from HEC-RAS (see Section 3.0) are processed in HEC-GeoRAS into inundation depth grids for each event ("depth grid"). For each alternative, the resulting depth grid is evaluated in HAZUS to produce an estimate of damages. These damages "with the selected improvement in place" are then compared to an estimate of damages in the existing condition, for the same storm event. The difference in damages is then the relative benefit for that particular flood control measure. Relative benefit is calculated for the 10% (10-year) and 1% (100-year) annual chance events.

For each HAZUS model run, the default Census and housing inventory databases are used. The USACE-Galveston District depth-damage curves are applied in deriving damage totals. Appendix H summarizes the 10% annual chance and 1% annual chance benefits for each alternative.

The benefits of each alternative should be evaluated in relation to the other alternatives. All alternatives are based on the completion of Alternative 1 Wilson Tract Diversion. This was assumed since Alternative 1 was under construction during the writing of this report.

7.2.2 Comparing Costs and Benefits

The benefit-cost ratio is the most commonly applied tool for determining the cost-effectiveness of undertaking an improvement, i.e. when the benefits expected exceed the cost of implementation, the project can be deemed viable. The benefit estimation methodology described in section 6.2.1 above assumes a relative benefit of a given improvement being in place. This is simply measured in terms of the difference in expected damages. It does not consider another critical criterion for determining the viability of an alternative: that the alternative considered cannot negatively affect or increase flooding at another location. In other words, it doesn't matter how much differential benefit can be derived if the alternative results in increased damage to some properties (see Figure 8). In fact, some alternatives considered may result in increased water surface elevations downstream. However, while this may still offer a

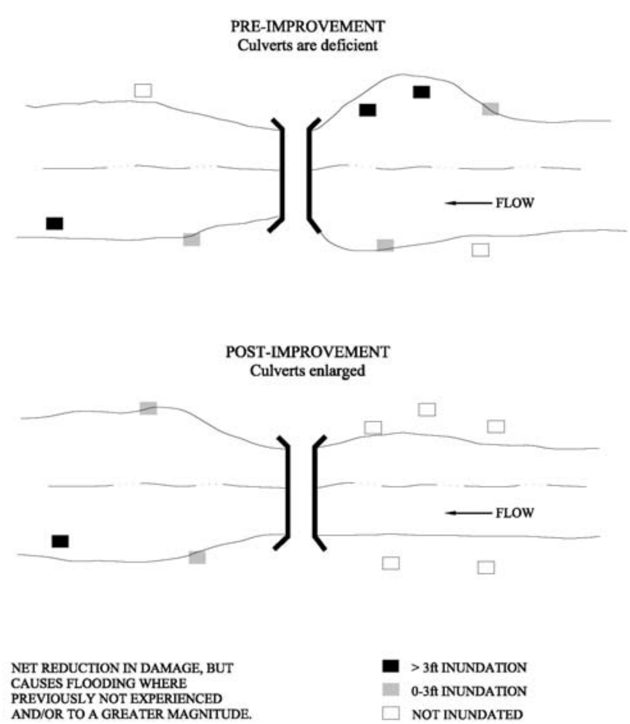


Figure 9: Undesirable alternative with positive benefit-cost ratio

benefit figure that exceeds the cost of implementation due to the particular spatial characteristics of structures and topography, clearly it is not good public policy to implement an alternative which causes more flooding than would otherwise exist. Therefore, in comparing costs and benefits, some alternatives which may appear to have a favorable ratio may be deemed less viable because they may result in increased flooding levels downstream. This increased flooding downstream is termed "adverse impact" in this report and is a factor considered in ranking the benefit and viability of each alternative in Chapter 7. In no case was the benefit great enough that the associated damages were deemed acceptable.

7.3 STRUCTURAL IMPROVEMENTS

This section provides a description and summary of estimated benefits and costs of the proposed alternatives to mitigate drainage and flooding issues in CCDD5. The figure below provides the general location of alternatives within CCDD5. Alternatives 7 and 9 are system-wide and not located on this figure.

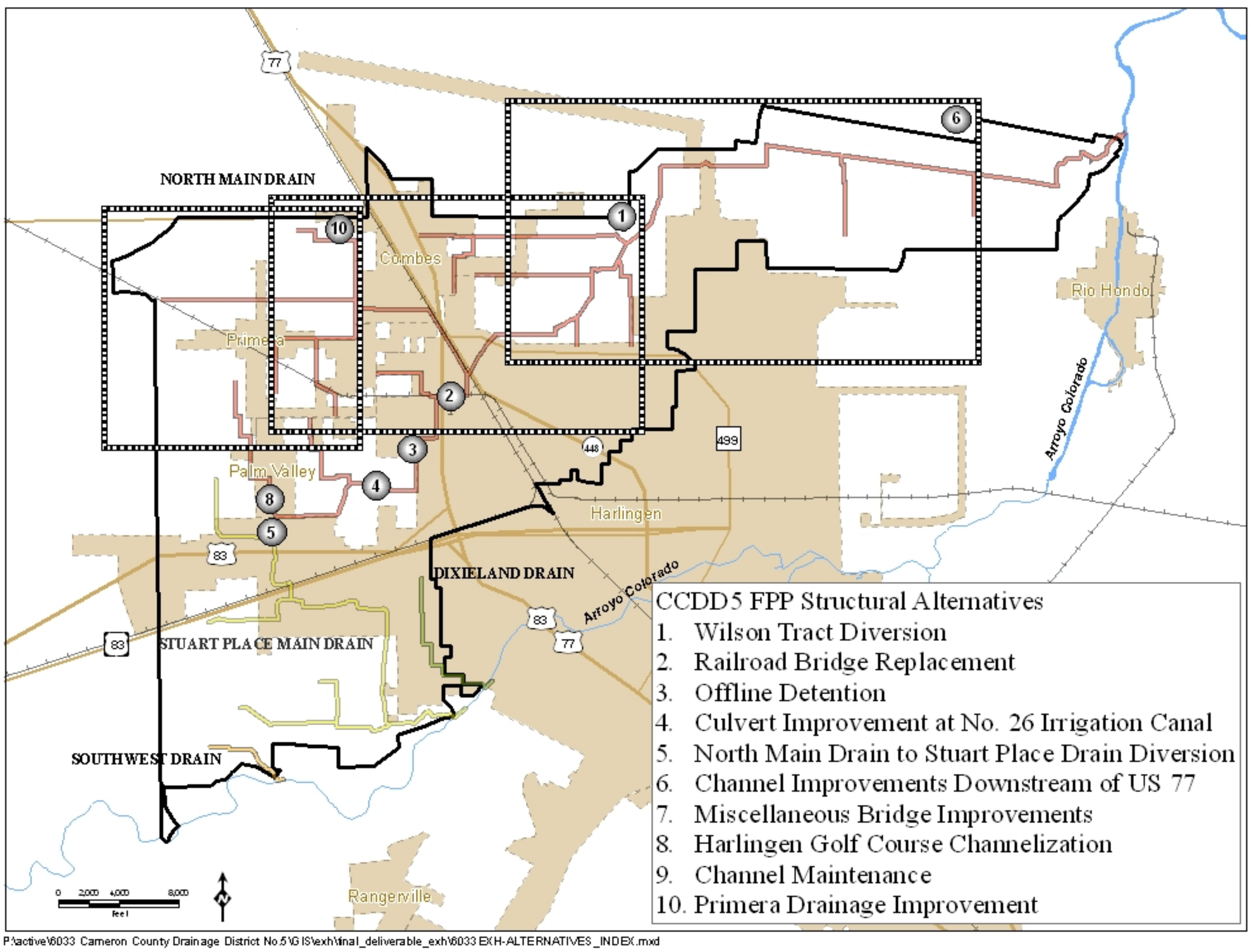


Figure 10. Location Map of CCDD5 Structural Alternatives

7.3.1 Alternative 1: Wilson Tract Diversion

The Town of Primera drains into the Wilson Tract Main (WTM), the Murphy-Primera Lateral and the Young Lateral. Flow in these laterals drain into the Primera Hand Lateral, which flows southeasterly into the North Main Drain main stem. The North Main Drain currently experiences a bottleneck downstream of its confluence with Primera Hand Lateral, just east of Loop 448.

Alternative 1 is also referred to as the Wilson Tract Diversion. This alternative includes the redirection, realignment, and extension of the Wilson Tract Main, the Primera Lateral, and the Young Lateral. The Wilson Tract Diversion will alleviate the bottleneck by redirecting flow from the Wilson Tract Main and Primera laterals north through the Young Lateral. First, a channel will be constructed extending from the intersection of the South Fork Lateral and Primera Lateral northward to the Wilson Tract Main. The new channel will direct water from the Murphy-Primera Lateral, the South Fork lateral and the Primera Lateral west of Cragon Road to the Wilson Tract Main Lateral. The channel will have a 12 foot bottom width, 44 foot top width, and 1.5 to 1 side slopes. Flow through the Primera Lateral east of Cragon Road will continue to towards the Young lateral, bypassing the Primera-Hand Lateral. A flow-limiting structure at the intersection of the Young and the Primera-Hand Lateral will be constructed to restrict flow into the Primera-Hand Lateral. The Young Lateral will be redirected to flow northward. The Young Lateral will be extended east in order to connect with the Wilson Tract Main Lateral east of US 77. The Young Lateral will connect with the Wilson Tract Main with a channel that will have at least 14 feet of depth, a 29 foot bottom width, 1.5 to 1 side slopes and a 14 foot wide shelf. Approximately 1,800 feet downstream of the confluence of the All States Lateral and the Wilson Tract Main Lateral, the Wilson Tract Main Lateral will be modified to turn south and connect with the David-Stephen Lateral, then to travel eastward to the North Main Drain main stem. The channel connecting the Wilson Tract Main Lateral with the David-Stephen Lateral will have at least 14 feet of depth, a 24 feet bottom width, 1.5 to 1 side slopes and a 14 feet wide shelf. A flow-limiting structure will be constructed to restrict flow from continuing east along the Wilson Tract Main Lateral. A detailed map of Alternative 1 is included as Exhibit 13 in Appendix A.

As part of Alternative 1, several culverts will need to be constructed where the new channel crosses existing roads. The table below lists the required culverts and their locations.

Table 21. Alternative 1 Proposed Culverts

Location	Proposed Culverts
Cragon Road at Wilson Tract Main Lateral	2-10ft x 10ft MBC, 96 LF ea.
Rio Rancho Road at Young Lateral	3-10ft x 10ft MBC, 34 LF ea.
Hand Road at Wilson Tract Main Lateral	3-10ft x 10ft MBC, 50 LF ea.
State Hwy 448 at connector between WTM and Young Laterals	6-8ft RCP, 96 LF ea.
US Hwy 77 at connector between WTM and Young Laterals	3-10 ft x 10ft MBC, 320 LF ea.
Kilbourn Road at connector between WTM and D&S Laterals	2-10ft x 10ft MBC, 55 LF ea.

Alternative 1 results in an average 10-year and 100-year water surface elevation reduction of approximately 0.5 feet, as shown in Exhibit 23 of Appendix A. The estimated benefit during the 10-year storm is $3.74

million and during the 100-year storm is $6.77 million. Calculations demonstrating the estimated benefit can be found in Appendix I. The Wilson Tract Diversion alternative includes channel improvements and bridge improvements.

The estimated cost of Alternative 1 is $3.8 million. Detailed estimates of costs can be found in Appendix H.

Construction on Alternative 1 began in 2007 in order to provide immediate relief to these areas. For this reason, all other alternatives analyzed in this study assume completion of Alternative 1.

7.3.2 Alternative 2: Railroad Bridge Replacement

Under existing conditions flow is obstructed by the skewed piers at the railroad bridge between US 77 and Loop 448 along the North Main Drain main stem. Alternative 2 proposes the replacement of the railroad bridge with two 10’ by 10’ box culverts, each with a length of 25 ft. A location map of Alternative 2 is found in Appendix A as Exhibit 14. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 2 will allow flow to pass unimpeded through the bridge in most storm events, which allows the North Main Drain to flow more effectively. This solution results in a water surface elevation reduction of slightly less than 1 foot for approximately 2 miles for the 2-, 5-, 10- and 25-year storm events. The improvements produce a minor benefit in the water surface elevation for the channel for all frequency events.

The estimated benefit of Alternative 2 during the 10-year storm is $710,000 and during the 100-year storm is there is no net benefit. Calculations supporting the estimated benefit can be found in Appendix I.

The estimated cost of Alternative 2 is $702,684. An estimate of costs can be found in Appendix H. It should be noted that costs do not include any considerations of additional efforts required in coordination with the railroad. This specific spur is not in use during part of the year, so it is assumed that construction would not interfere with normal rail traffic.

7.3.3 Alternative 3: Offline Detention

Under existing conditions, the North Main Drain experiences out of channel flooding at various locations along the reach during the 5-year storm event. Alternative 3 proposes the addition of offline detention along the North Main Drain main stem. The proposed detention pond would be situated at an existing borrow pit located south of FM 2994 and west of US 77, on land owned by CCDD5. In its existing condition, the site has a storage volume of 43.7 acre-feet. Alternative 3 is illustrated in Appendix A as Exhibit 15. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

The existing pond would be excavated to a depth matching the channel flow line of the adjacent North Main Drain main stem, enabling an increase in pond volume from 43.7 acre-feet to 100 acre-feet. The pond is assumed to have an inlet-outlet structure that will not restrict the flow to the pond. The proposed pond would have 3:1 side slopes.

The estimated benefit of Alternative 3 for a 10-year storm is $1.21 million and for a 100-year storm is $1.59 million. Water surface elevations adjacent to the pond are reduced. The 25-year water surface reduction is less than one foot. Calculations supporting the estimated benefit can be found in Appendix I.

The estimated cost of Alternative 3 is $704,498 as summarized in Appendix H.

7.3.1 Alternative 4: Culvert Improvement at No. 26 Irrigation Canal

Alternative 4A considers the replacement of the existing two 5' x 5' culvert boxes, 72 feet long at an elevation of 31.8 feet with one 10' x 10' culvert box, 72 feet long with an invert at an elevation of 28 feet (i.e., 3.8' lower than exists currently).

The estimated benefit of Alternative 4 during the 10-year storm is $90,000 and during the 100-year storm is $4.08 million. The 25-year water surface reduction is 0.5 feet upstream of Irrigation Canal. Also in this instance, damage is caused downstream as a result of the alternative. Calculations describing the estimated benefit can be found in Appendix I.

The estimated cost of Alternative 4 is $154,009. Calculations of costs are included in Appendix H.

7.3.2 Alternative 4A: Culvert Removal at No. 26 Irrigation Canal

The North Main Drain main stem crosses under the No. 26 irrigation canal, approximately 90 ft downstream of Chester Park Road. The existing crossing consists of two 5'x 5' box culverts perched approximately 4 feet above the flow line of the channel. The perched configuration causes water upstream of the canal to back up in the channel until the water surface elevation reaches the level of the culvert invert. In this manner, the channel functions as a linear detention pond in low events, as the bottom few feet of the channel provide flood storage.

Alternative 4 proposes the removal of the two 5'x 5'perched culvert boxes under the No. 26 Irrigation Canal, along with the relocation and reconstruction of the irrigation canal to a terminus south of the North Main Drain crossing. In this configuration, the North Main Drain functions as an open channel at this location. Alternative 4 is illustrated as Exhibit 16 in Appendix A. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

The estimated benefit for the 10-year storm is $40,000 and for the 100-year storm is 4.61 million. The 25-year water surface reduction is 0.5 feet upstream of Irrigation Canal. Calculations supporting this benefit can be found in Appendix I. In any storm event, downstream water surface elevations are increased, and therefore this alternative must be very carefully considered for implementation.

The estimated cost of Alternative 4A is $426,245. This estimate does include the additional cost for relocation of the pump station, from the existing terminus of the irrigation canal north of the North Main Drain main stem to the new terminus of the irrigation canal, just south of the main stem. Calculations for cost estimate can be found in Appendix H.

7.3.3 Alternative 5: Diversion from North Main to Stuart Place Main

North Main Drain and Stuart Place Main are connected by a trapezoidal channel that begins at the North Main Drain main stem south of the Harlingen Country Club Golf Course, crosses under Acacia Drive, and connects with the Stuart Place Main upstream of US 83. Currently, during the 100-year event the Acacia Connector will divert approximately 30% of the flow upstream of the connector from North Main Drain to Stuart Place Main. Alternative 5 is illustrated as Exhibit 17 in Appendix A. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 5 proposes to re-route all of the flow from the North Main Drain upstream of the Acacia Connector to Stuart Place Main through a diversion structure. This structure will include an overflow weir to allow emergency overflows to return to North Main Drain to prevent high water surface elevations along Stuart Place Main. It should be noted that the high tailwater condition present at Chester Park (see Alternative 4 above) assists in the diversion contemplated here with Alternative 5.

The estimated benefit of Alternative 5 for the 10-year storm $260,000 and for the 100-year storm is $3.19 million. However, this alternative would increase flooding on Stuart Place Main with only minor reductions in water surface elevation along the North Main Drain. Calculations of the estimated benefit can be found in Appendix I.

The estimated cost of Alternative 5 is $49,285, including construction of the diversion structure. A detailed cost estimate can be found in Appendix H.

7.3.4 Alternative 6: Channel Improvements Downstream of US 77

Under existing conditions (both with and without Alternative 1 in-place), North Main Drain generally has the worst flooding problems between Montezuma Road and Searcy Ranch Road. In this section, the channel has an average, but not uniform, slope of 0.022%. This channel reach has been over-excavated during maintenance, resulting in a channel bed that increases and decreases, with a slope ranging from 0.016% to 0.64%. The lower event storms (2-year, 5-year, 10-year) have a relatively high water surface elevation in this reach due to the non-uniform slope of the channel (over-excavated in some reaches). This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 6 includes a series of channel and bridge improvements along an approximately 15 mile stretch of the North Main Drain main stem, from Montezuma Road to downstream of Searcy Ranch Road. Under this alternative, the channel would be improved to a trapezoidal cross-section with a 55 foot bottom width and 1.5 to 1 side slopes. The flow line would have a uniform slope of 0.022%. Several bridge improvements will be required. The Searcy Ranch Road bridge deck would be raised to reduce flow restriction. The bridge deck at the FM 507 crossing would be raised 2.5 feet. Right-of-way requirements will increase by approximately 50 feet, to approximately 130 feet total width.

These channel improvements lower the water surface elevation of the North Main Drain main stem and its adjacent tributaries. These tributaries are significantly influenced by backwater, which will be reduced with improved hydraulic capacity along this reach of the main stem. Alternative 6 is illustrated as Exhibit 18 in Appendix A. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

The estimated benefit of Alternative 6 for the 10-year storm is $3.55 million and during the 100-year storm is $4.48 million. The flow line improvements will lower the water surface elevation of the lower event storms (2-year, 5-year, 10-year) by as much as 4.5 feet. This benefit is shown in the Alternative 6 water surface profile included as Exhibit 24 in Appendix A. Calculations for the estimated benefit can be found in Appendix I.

The estimated cost of Alternative 6 is $3,646,319. Calculations to estimate this cost can be found in Appendix H.

7.3.5 Alternative 7: Miscellaneous Bridge Improvements

Alternative 7 identifies bridges and culverts throughout the study area that restrict flow during and following storm events. Exhibit 19 in Appendix A shows the location of these structures.

Three bridges were identified for improvement in North Main Drain watershed:

- North Main Drain main stem at Stuart Place Road;
- Young Lateral at Cragon Road; and,
- Murphy Lateral at Primera Road.

Flow restriction at these bridges and culverts causes flooding and backwater issues. However, these are minor relative to the areas and structures studied in the other alternatives.

Along Dixieland Main, two structures were identified for improvement to alleviate flooding along the reach between Lincoln Street and Bothwell Road:

- Dixieland Main main stem at Bothwell Road, and
- Dixieland Main main stem between Bothwell Road and Garrett Road.

Under existing conditions flow is obstructed at the 60' diameter RCP culvert at Bothwell Road due to sedimentation build up. Regular maintenance is recommended. Cleaning out this culvert lowers the water surface elevation upstream of Bothwell Road significantly: the 2-yr water surface elevation is lowered by 0.5 ft, the 5-yr water surface elevation by 2 ft, and the 10-yr water surface elevation by 1 ft upstream. The 25-yr and 100-yr water surface elevations are not affected.

Removal of the 60-inch diameter RCP culvert between Garrett Road and Bothwell Road will have minor impacts on the water surface elevation under the existing conditions. The impact on water surface elevations due to the removal of this culvert would have an impact of less than 0.2 ft for the 2-, 5-, 10-, and 25-yr events. If the currently obstructed culvert at Bothwell Road were to be cleared (as described above) the impacts of removing this culvert could be much more significant.

7.3.6 Alternative 8: Harlingen Golf Course Channelization

The Harlingen Country Club is served by the most upstream reach of the North Main Drain. A 30-inch pipe with a conveyance capacity of less than the 2-year storm runs under the golf course. This is a significant impediment to flows, resulting in flooding upstream of the golf course. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 8 proposes the replacement of the existing pipe with an open trapezoidal channel. The channel would tie into the existing channel configuration, upstream and downstream of the removed culvert pipe. This channel, with a 12 foot bottom width and approximately 1.5:1 side slopes, would provide a 25-year level of service through the golf course. An illustration of Alternative 8 is included as Exhibit 20 in Appendix A.

The estimated cost of Alternative 8 is $590,871. This estimate includes demolition of culverts, re-vegetation, and right-of-way costs. A detailed cost estimate can be found in Appendix I.

However, benefits are only realized upstream of the golf course. Flows are increased downstream of the golf course, resulting in a negation of any benefits realized upstream. Calculations describing these estimated benefits can be found in Appendix H.

7.3.7 Alternative 9: Channel Maintenance

In recent years, CCDD5 has spent approximately $300,000 annually on channel maintenance. Channel maintenance includes mowing banks of channels to reduce channel roughness and removing debris to reduce the risk of the debris becoming an obstruction to the channel. Maintaining vegetation along the banks is also important as tall grass and brush impedes flow into the ditches, increasing ponding. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 9 proposes varying the channel maintenance frequency of North Main Drain by simulating both a doubling and halving of the frequency. Variation in channel maintenance frequency is modeled through varying the channel Manning's roughness coefficient, a measure of the ground surface roughness. This allows the hydraulic model sensitivity to height and type of vegetation to be analyzed. To assess the effects of doubling the maintenance frequency, the Manning's roughness coefficient was reduced from 0.045 to 0.35 to reflect the decrease in ground surface roughness simulating a smoother surface indicative of increased maintenance. To assess the effects of halving the maintenance frequency, the Manning's roughness coefficient for the channel was increased from 0.045 to 0.055, to reflect the increase in ground surface roughness, simulating taller grass and debris, indicative of a reduced maintenance program. Exhibit 21 in Appendix A includes photographs to represent ground conditions corresponding to the Manning's n values.

For all events doubling the maintenance frequency results in reductions in water surface elevation along the North Main Drain main stem except upstream of the skewed railroad bridge east of US 77 for the 100-year event. Water surface elevations for all storm events are reduced for all storm events except for the 100-year event along Carter's Lateral. The benefit of doubling the maintenance budget under the 10-year storm is estimated as a savings of $1.5 million in damages. The cost of doubling the maintenance frequency only includes the maintenance work, an annual expenditure increase from $300,000 to $600,000. Details of benefit calculations are included in Appendix I.

There are no flood damage cost benefits with the maintenance frequency halved as water surface elevations are increased during storm events. The cost of halving the maintenance frequency includes an annual channel maintenance expenditure of $150,000 and the cost of flood damages incurred during storm events. The estimated cost for damages under the 10-year storm is $768,000. Cost calculations are included in Appendix H.

7.3.8 Alternative 10: Primera Drainage Improvement

The Town of Primera experiences local drainage issues as a result of poor conveyance through culverts at road crossings. Most frequently, the community experiences ponding, as was described at a public input meeting held by CCDD5 at Primera Town Hall on October 19, 2006. Alternative 10 represents an effort to facilitate local drainage into District channels by reducing water surface elevations in the channels. Generally, this can be accomplished by improving channel hydraulic capacity. This alternative is analyzed assuming the completion of the Alternative 1 Wilson Tract Diversion.

Alternative 10 includes approximately 5,900 ft of channel excavation and 6 culvert replacements along North Main Drain ditches in the Primera area. Channel excavation is considered along the Wilson Tract Lateral from Stuart Place Road to Cragon Road, and along Murphy-Primera Lateral from 500' downstream of Wilcox Road to the confluence of the Murphy-Primera Lateral with the South Fork Lateral. Location of proposed structures and channel improvements are illustrated in Exhibit 22 of Appendix A. The table below lists the proposed culverts along the Wilson Tract Main Lateral and the South Fork Lateral.

Table 22. Alternative 10 Proposed Culvert Replacements

Location	Existing Culverts	Proposed Culverts
Alonzo Road at Wilson Tract Main Lateral	1-2.5ft RCP, 50 LF ea.	2-5' x 5' MBC, 50 LF ea.
Stuart Place Road at Wilson Tract Main Lateral	1-3ft RCP, 41 LF ea.	2-5' x 5' MBC, 41 LF ea.
Wilcox Road at Wilson Tract Main Lateral	1-1.5ft RCP, 27 LF ea.	2-5' x 5' MBC, 27 LF ea.
Railroad Crossing at South Fork Lateral	1-1.5ft RCP, 44 LF ea.	2-4' x 4' MBC, 44 LF ea.
Field Crossing near Carver Road at South Fork Lateral	1-1.5ft RCP, 40 LF ea.	2-4' x 4' MBC, 40 LF ea.
Primera Road at South Fork Lateral	1-2ft RCP, 1-2.5ft RCP, 52 LF ea.	2-4' x 4' MBC, 52 LF ea.

The estimated benefit achieved through the implementation of Alternative 10, included in Appendix I, during the 10-year storm is $2.27 million; for the 100-year storm the benefit is estimated at $1.77 million. Upsizing these culverts improves the hydraulic efficiency in the channels draining Primera. However, these improvements allow runoff to drain more rapidly from Primera, resulting in increased flows on flood-prone areas downstream. For this reason, Alternative 10 would be viable only in conjunction with Alternative 6. Alternative 6 improves the hydraulic capacity along the North Main Drain main stem downstream of US 77, which would in turn enable the channels to accommodate the increased flows due to Alternative 10. A detailed series of estimates of the construction costs for Alternative 10 is provided in Appendix H.

In total, the estimated cost for these six culvert replacements and associated channel improvements to implement Alternative 10 is $534,301. This cost includes channel excavation and culvert construction. Calculations and detail for the cost estimate of Alternative 10 is found in Appendix H.

7.4 NON-STRUCTURAL FLOOD PROTECTION MEASURES

Along with the channel improvements, culvert upgrades and detention pond construction alternatives discussed above, the District can cost-effectively implement a series of non-structural measures as part of its overall flood protection planning efforts. These include: addressing illegal dumping, expanding the

District's rainfall and streamflow gaging network, establishing coordinated stormwater runoff control policies among jurisdictions, and acquisition of flood prone properties. Each is discussed in more detail in the sections below.

7.4.1 Addressing Illegal Dumping

Disposal of debris into the drainage ditches creates blockages, which increases flooding. This likely seems self-evident to the reader, but remarkably is an ongoing, chronic problem with severe consequences. Addressing the problem involves a regional, three-pronged approach: promoting awareness, expanding legal disposal opportunities, and enforcement. Coordination with the County, cities within the region, neighborhood associations, solid waste providers, the school districts, and the Lower Rio Grande Valley Development Council (LRGVDC) should be sought in an initiative to leverage resources and employ a coordinated approach. For instance, coordinating a media campaign involving radio, newspaper, movie theater screening ads, and a poster program at area middle schools would be a more cost effective means of reaching the regional audience than individual local entity efforts by themselves. Similarly, expanding legal disposal opportunities will require coordination with local solid waste service providers, and if coordinated with other entities in the area, the effort can reach a larger geographic area. Ultimately, tracking violators and prosecuting these offenses is necessary to deter the crime, but will require coordination among area law enforcement agencies.

7.4.2 Install Streamflow Gaging Network

CCDD5 is currently installing a network of streamflow gages along the CCDD5 drainage ditches. The streamflow gages will tie into the existing telemetry system that collects rainfall gage data. This data is made public through Cameron County Irrigation District 1. Data collected from the rainfall and streamflow gages may be used to monitor flooding conditions in a flood-alert system. This information serves two purposes. First, it brings critical information to the District about potential problems, and allows the District and other entities to see the problems in one central location simultaneously, as the issues develop. Second, the data collected from this network creates a record to monitor the behavior of the system in correlation with rainfall. This then enables the District to continually refine its models of the ditch system.

7.4.3 Coordinated Stormwater Management Policy

The Lower Rio Grande Valley is experiencing tremendous growth. Without management practices, development increases impervious cover, which increases rainfall runoff, raises water surface elevations and increases flooding. Coordination with the Towns of Primera, Combes and Palm Valley, City of Harlingen, Cameron County and CCDD5 is necessary to develop practical and enforceable policy. The rules developed under such an initiative may require limits on impervious cover and/or require on-site detention for future developments.

7.4.4 Voluntary Acquisition of Flood Prone Areas

Removing residents from flood-prone areas through the purchase of such properties reduces flooding risk. Buying flood prone structures through a voluntary acquisition or relocation program is a common practice among communities. The estimated cost of this solution will vary according to property value and cost of demolition. This alternative will potentially reduce the amount of property damage caused by flooding. Also, as undeveloped District-owned property, this land may serve as minor detention, recreational areas, and wildlife habitat. Funding may be available through the Flood Mitigation Assistance Program (FMA) for a targeted, voluntary acquisition and relocation program. CCDD5 would need to work through an NFIP-community sponsor to utilize these funds, since the District is not an NFIP community in and of itself.

7.4.5 Public Education Campaign: The Benefits of Purchasing Flood Insurance

The reality of floodplain management in the Lower Rio Grande Valley is that large areas are very susceptible to flooding, due to the flat topography and despite the largest capital improvements projects. With this understanding must come a recognition of risk, and to guard against that risk individuals in the community should consider the value of purchasing flood insurance. As a participant with other community partners, the District should engage in a public education campaign to promote awareness of the National Flood Insurance Program (NFIP). The Regional NFIP Coordinator can be contacted at 956-421-3214.

8.0 FLOOD PROTECTION PLAN

The response to these flooding issues is directed by a series of policy goals, analyses and actions, as formulated below. A goal is a desired end or outcome. The analysis discusses the technical basis behind the goal, and supplies the impetus to the individual actions. The actions are specific projects, programs or activities which are recommended for implementation in order to achieve the goal. Taken all together, these goals represent the long term approach that CCDD5 and its partners in floodplain management must take in order to address the flood hazard present along the District's network. In short these goals are as follows:

Goal 1: Proactively address flood problem areas with targeted improvements that consider the entire District's service area
Goal 2: Ensure that new development does not adversely affect property downstream
Goal 3: Upstream of the District's ditch network, local development should ensure positive drainage to the District's network; the District should ensure the lowest possible tailwater conditions to facilitate local drainage
Goal 4: Protect and enhance available storage in the system
Goal 5: Actively inform the community of the risk of flooding
Goal 6: Aggressively pursue a regional approach to curb illegal dumping
Goal 7: Update and refine the Flood Protection Plan on a bi-annual basis

The following sections describe important analyses, considerations and actions to be taken in furthering each goal.

8.1 FURTHERING FLOOD PROTECTION GOALS

<u>Goal 1</u>**: Proactively address flood problem areas with targeted improvements that consider the entire District's service area**. The engineering analysis has identified several structural improvement options that can provide immediate benefit to the District. The following actions can be taken towards implementing this goal:

- <u>Action 1.1</u> – Complete construction of Alternative 1, Wilson Tract Diversion.
- <u>Action 1.2</u> – Design and construct a detention pond at the borrow pit located at south of FM 2994 and west of US 77 (Alternative 3).
- <u>Action 1.3</u> – Develop a multi-year implementation program to construct targeted channel and bridge improvements on the North Main Drain identified in Alternatives 6 and 7.
- <u>Action 1.4</u> – Assist the Town of Primera in placing the identified improvements of Alternative 10 in their Comprehensive Plan in order to leverage developer participation.
- <u>Action 1.5</u> – Begin discussions with the railroad to cooperatively replace the skewed piers with box culverts (Alternative 2).

<u>Goal 2</u>: Ensure that new development does not adversely affect property downstream. This represents a "good neighbor" policy inasmuch as it reflects a very real limit on available conveyance capacity within the District. The following actions can be taken towards implementing this goal:

- Action 2.1 – Evaluate the feasibility of requiring on-site detention for at least the 10% annual chance event to mitigate the impacts of site-specific development.
- Action 2.2 – Coordinate with local governments to establish common standards and hydrologic and hydraulic methods and assumptions.

<u>Goal 3</u>: Upstream of the District's ditch network, local development should ensure positive drainage to the District's network; the District should ensure the lowest possible tailwater conditions to facilitate local drainage. Many nuisance drainage problems can be alleviated if good, positive drainage exists. For CCDD5's part, measures to reduce any tailwater in the ditches will further this goal, and the following specific action:

- Action 3.1 – Following the implementation of Action Item 1.3 (Alternative 6), begin a phased program to implement the recommended channel and culvert improvements of Alternative 10.

<u>Goal 4</u>: Protect and enhance available storage in the system. Valley storage in the ditch network is a critical resource from a hydraulic perspective.

- Action 4.1 – Acquire (by right-of-way or easement) areas which are subject to high headwater conditions and where analysis indicates that increasing conveyance at that location will result in adverse downstream impacts.
- Action 4.2 – Acquire sufficient right-of-way to introduce a bench channel section in implementing Alternative 6, and in other areas where feasible.

<u>Goal 5</u>: Actively inform the community of the risk of flooding. It is important for the District to actively inform the community about the nature of flood risk, and the limits of what the District can do to mitigate that risk. The following actions can be taken towards implementing this goal:

- Action 5.1 – The District's ability to provide higher levels of flood protection are limited by regional topography, available right-of-way, and existing encroachment into the floodplain. While especially true for large events (100-year and 25-year), the improvements contemplated by the District will have significant effect on smaller events (2-year through 25-year). The current level of risk for larger events should be made freely available through dissemination of floodplain maps, both paper and digital.
- Action 5.2 – Make an initial presentation to the Chamber of Commerce, and follow up with "annual update" presentations, or contribute articles to the Chamber's newsletter with updates on CCDD#5 activities.
- Action 5.3 – Identify neighborhood leaders in flood-prone neighborhoods and develop a specific outreach campaign with their guidance.
- Action 5.4 – Work with private industry and other stakeholders to develop and implement a program to distribute NOAA All Hazards Weather Radios to the public.

- Action 5.5 – Working with other authorities, develop a specific plat note requirement to explain the limitations of flood protection in the Lower Rio Grande Valley.

Goal 6: **Aggressively pursue a regional approach to curb illegal dumping**. This is probably the most preventable cause of flooding, but will require a coordinated effort with other entities, and a multi-pronged approach. The following actions can be taken towards implementing this goal:

- Action 6.1 – Recognizing that the illegal dumping problem is a regional issue, work with the Lower Rio Grande Valley Development Council to find the best long-term solutions.
- Action 6.2 – Pursue grant funded opportunities through TCEQ and LRGVDC to host "clean-up" activities.
- Action 6.3 – Develop a public awareness program using signs at access points to CCDD#5 ditches. Slogans and posters in two languages could be developed by working with area middle schools (school-wide competition, for example). Examples of such slogans: "You dump, we have to pump" and "Basura tirada, casa inundada"
- Action 6.4 – Install gates at access points
- Action 6.5 – Reach out to neighborhood leaders to explain the issue and risks at hand, and solicit their input on ways to curb the problem and raise awareness.

Goal 7: **Update and refine the Flood Protection Plan on a bi-annual basis**. Over time, the conditions in the watershed will change and the Flood Protection Plan will need to be updated, and be viewed as a living document. The following actions can be taken towards implementing this goal:

- Action 7.1 – Continue the installation of telemetry-based gages to monitor flow, stage, and velocity.
- Action 7.2 – Perform a model update on a bi-annual basis to incorporate new development and calibration data, if available.
- Action 7.3 – Assess and prioritize the remaining construction projects, knowing that many conditions in the watershed will change over time.

8.2 PRIORITIZATION, IMPLEMENTATION AND PHASING

Each alternative that was studied offers specific benefits and costs, as well as specific policy implications. However, there are generally many other factors which should be considered in prioritizing and selecting various alternatives beyond the benefit-cost ratio. Initially, the consultant presented the Advisory Committee a scoring matrix exercise involving multiple factors. The Advisory Committee felt that a detailed scoring matrix was not necessary, because the ranking was mostly a function of immediate dollar benefit and cost-effectiveness. Therefore, the scoring matrix exercise was put aside and a simpler method was employed.

8.2.1 Ranking Method

Each of the alternatives were entered into the ranking process, with the exception of Alternative 7 (benefits not estimated due to very minor effect relative to other alternatives), and Alternative 9 (benefits estimated, but could only be measured relative to each other, not relative to current maintenance program[2]). For each of the alternatives that were entered into the ranking process, a benefit-cost ratio (BCR) was calculated, as seen in the following table:

Table 23: Benefit-Cost Summary of Alternatives

RANKING	PROJECT	10-year BENEFIT	100-year BENEFIT	COST	BCR (10-year)	BCR (100-year)	ADVERSE IMPACTS
1	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	$3,740,000	$6,770,000	$3,817,000	0.98	1.77	NO
2	Alternative 2: Railroad Bridge Replacement	$710,000	-$120,000	$702,684	1.01	-0.17	NO
3	Alternative 3: Offline Detention Basin	$1,210,000	$1,590,000	$704,497	1.72	2.26	NO
	Alternative 4: Culvert **Improvement** at Chester Park Irrigation Canal	$90,000	$4,080,000	$154,009	0.58	26.49	YES
	Alternative 4a: Culvert **Removal** at Chester Park Irrigation Canal	$40,000	$4,610,000	$426,244	0.09	10.82	YES
	Alternative 5: Diversion to Stuart Place Main	$260,000	$3,190,000	$49,285	5.28	64.73	YES
4	Alternative 6: Proposed Channel Improvement	$3,550,000	$4,480,000	$3,646,319	0.97	1.23	NO
	Alternative 8: Harlingen Country Club Golf Course Channelization	-$100,000	-$820,000	$590,871	-0.17	-1.39	YES
5	Alternative 10: Primera Improvements	$2,270,000	$1,770,000	$534,301	4.25	3.31	NO

Table 25 also indicates a review of the criterion that no adverse impacts be created with each alternative. The alternatives were then ranked in three ways in order to reflect the Advisory Committee's priority criteria: by benefit-cost ratio, by 10-year benefit, and by 100-year benefit. Following these rankings, a composite score was developed to provide an overall ranking. A rank of 1 received a score of 5, a rank of 2 received a score of 4, and so forth. Alternatives which result in adverse impacts received a score of zero, and therefore fell out of the rankings. The following table summarizes the composite scoring exercise:

Table 24: Scoring and Ranking of Alternatives

Sorted by 10-year Benefit

RANKING	PROJECT	10-year BENEFIT	100-year BENEFIT	COST	BCR (10-year)	BCR (100-year)	ADVERSE IMPACTS
1	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	$3,740,000	$6,770,000	$3,817,000	0.98	1.77	NO
2	Alternative 6: Proposed Channel Improvement	$3,550,000	$4,480,000	$3,646,319	0.97	1.23	NO
3	Alternative 10: Primera Improvements	$2,270,000	$1,770,000	$534,301	4.25	3.31	NO
4	Alternative 3: Offline Detention Basin	$1,210,000	$1,590,000	$704,497	1.72	2.26	NO
5	Alternative 2: Railroad Bridge Replacement	$710,000	-$120,000	$702,684	1.01	-0.17	NO
	Alternative 5: Diversion to Stuart Place Main	$260,000	$3,190,000	$49,285	5.28	64.73	YES
	Alternative 4: Culvert **Improvement** at Chester Park Irrigation Canal	$90,000	$4,080,000	$154,009	0.58	26.49	YES
	Alternative 4a: Culvert **Removal** at Chester Park Irrigation Canal	$40,000	$4,610,000	$426,244	0.09	10.82	YES
	Alternative 8: Harlingen Country Club Golf Course Channelization	-$100,000	-$820,000	$590,871	-0.17	-1.39	YES

[2] As an aside recommendation, since some differential benefit between the 10-year and 100-year scenarios was realized, it may be worth further study to determine if reduced maintenance in the upper reaches actually provides benefit by increasing storage. This differential maintenance was not the initial thrust of Alternative 9, but the results of Alternative 9 appear to indicate that with some refinement, additional benefit may be realized.

Table 25: Scoring and Ranking of Alternatives Continued

Sorted by 100-year Benefit

RANKING	PROJECT	10-year BENEFIT	100-year BENEFIT	COST	BCR (10-year)	BCR (100-year)	ADVERSE IMPACTS
1	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	$3,740,000	$6,770,000	$3,817,000	0.98	1.77	NO
	Alternative 4a: Culvert **Removal** at Chester Park Irrigation Canal	$40,000	$4,610,000	$426,244	0.09	10.82	YES
2	Alternative 6: Proposed Channel Improvement	$3,550,000	$4,480,000	$3,646,319	0.97	1.23	NO
	Alternative 4: Culvert **Improvement** at Chester Park Irrigation Canal	$90,000	$4,080,000	$154,009	0.58	26.49	YES
	Alternative 5: Diversion to Stuart Place Main	$260,000	$3,190,000	$49,285	5.28	64.73	YES
3	Alternative 10: Primera Improvements	$2,270,000	$1,770,000	$534,301	4.25	3.31	NO
4	Alternative 3: Offline Detention Basin	$1,210,000	$1,590,000	$704,497	1.72	2.26	NO
5	Alternative 2: Railroad Bridge Replacement	$710,000	-$120,000	$702,684	1.01	-0.17	NO
	Alternative 8: Harlingen Country Club Golf Course Channelization	-$100,000	-$820,000	$590,871	-0.17	-1.39	YES

Sorted by 10-year BCR

RANKING	PROJECT	10-year BENEFIT	100-year BENEFIT	COST	BCR (10-year)	BCR (100-year)	ADVERSE IMPACTS
	Alternative 5: Diversion to Stuart Place Main	$260,000	$3,190,000	$49,285	5.28	64.73	YES
1	Alternative 10: Primera Improvements	$2,270,000	$1,770,000	$534,301	4.25	3.31	NO
2	Alternative 3: Offline Detention Basin	$1,210,000	$1,590,000	$704,497	1.72	2.26	NO
3	Alternative 2: Railroad Bridge Replacement	$710,000	-$120,000	$702,684	1.01	-0.17	NO
4	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	$3,740,000	$6,770,000	$3,817,000	0.98	1.77	NO
5	Alternative 6: Proposed Channel Improvement	$3,550,000	$4,480,000	$3,646,319	0.97	1.23	NO
	Alternative 4: Culvert **Improvement** at Chester Park Irrigation Canal	$90,000	$4,080,000	$154,009	0.58	26.49	YES
	Alternative 4a: Culvert **Removal** at Chester Park Irrigation Canal	$40,000	$4,610,000	$426,244	0.09	10.82	YES
	Alternative 8: Harlingen Country Club Golf Course Channelization	-$100,000	-$820,000	$590,871	-0.17	-1.39	YES

Sorted by 100-year BCR

RANKING	PROJECT	10-year BENEFIT	100-year BENEFIT	COST	BCR (10-year)	BCR (100-year)	ADVERSE IMPACTS
	Alternative 5: Diversion to Stuart Place Main	$260,000	$3,190,000	$49,285	5.28	64.73	YES
	Alternative 4: Culvert **Improvement** at Chester Park Irrigation Canal	$90,000	$4,080,000	$154,009	0.58	26.49	YES
	Alternative 4a: Culvert **Removal** at Chester Park Irrigation Canal	$40,000	$4,610,000	$426,244	0.09	10.82	YES
1	Alternative 10: Primera Improvements	$2,270,000	$1,770,000	$534,301	4.25	3.31	NO
2	Alternative 3: Offline Detention Basin	$1,210,000	$1,590,000	$704,497	1.72	2.26	NO
3	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	$3,740,000	$6,770,000	$3,817,000	0.98	1.77	NO
4	Alternative 6: Proposed Channel Improvement	$3,550,000	$4,480,000	$3,646,319	0.97	1.23	NO
5	Alternative 2: Railroad Bridge Replacement	$710,000	-$120,000	$702,684	1.01	-0.17	NO
	Alternative 8: Harlingen Country Club Golf Course Channelization	-$100,000	-$820,000	$590,871	-0.17	-1.39	YES

Composite Scoring

PRIORITY	PROJECT	RANK BY 10-year BENEFIT	RANK BY 100-year BENEFIT	RANK BY 10-year BCR	RANK BY 100-year BCR	(Rank 1=5 pts, Rank 2=4pts, etc.)
1	Alternative 10: Primera Improvements	3	3	1	1	16
2	Alternative 1: Reversal of Flow Direction Towards Proposed Channel Connector	1	1	4	3	15
3	Alternative 3: Offline Detention Basin	4	4	2	2	13
4	Alternative 6: Proposed Channel Improvement	2	2	5	4	11
5	Alternative 2: Railroad Bridge Replacement	5	5	3	5	6

Thus, based on the scoring exercise, the top priority project for the District to complete is Alternative 1: Wilson Tract Connections and Improvements. The least priority project to complete is Alternative 2: Railroad Bridge Replacement. The following section describes the Funding and Priority Plan for the recommended actions.

8.3 FUNDING AND PRIORITY PLAN

The mechanisms for funding of the recommended actions vary and in some instances are specific to the action. The intent of this section is to identify funding sources for each action, as well as strategies to leverage funding mechanisms, and provide an idea of implementation timeline.

Table 25: Action Funding and Priority Plan

Action Item	Funding Source / Regulation / Effort Type	Priority
Action 1.1 : Complete construction of Alternative 1, Wilson Tract Diversion	Capital Budget	Immediate
Action 1.2 : Design and construct a detention pond at the borrow pit located south of FM 2994 and west of US 77 (Alternative 3)	Capital Budget; Hazard Mitigation Grant Funds	Immediate
Action 1.3 : Develop a multi-year implementation program to construct targeted channel and bridge improvements on the Nort Main Drain identified in Alternatives 6 and 7.	Capital Budget; Hazard Mitigation Grant Funds	Short-Term
Action 1.4 : Assist the Town of Primera in placing the identified improvements of Alternative 10 into their Comprehensive Plan, in order to leverage developer participation (see Item 3.1)	Capital Budget; Hazard Mitigation Grant Funds; Public-Private Partnerships; Impact Fees	Short-Term; Implement projects after Alternative 6 (Action 1.3)
Action 1.5 : Begin disussions with the railroad to cooperatively replace the skewed piers with box culverts (Alternative 2)	Capital Budget /both parties	Long-Term
Action 2.1 : Evaluate the feasibility of requiring on-site detention for at least the 10% annual chance event to mitigate the impacts of site-specific development	Joint task force of City, District, County, Towns	Immediate
Action 2.2 : Coordinate with local governments to establish common standards and hydrologic and hydraulic methods and assumptions	Joint task force of City, District, County, Towns	Immediate
Action 3.1 : Following the implementation of Action Item 1.3 (Alternative 6), begin a phased program to implement the recommended channel and culvert improvements of Alternative 10.	Capital Budget; Hazard Mitigation Grant Funds; Public-Private Partnerships; Impact Fees	Long-Term
Action 4.1 : Acquire (by right-of-way or easement) areas which are subject to high headwater conditions and where analysis indicates that increasing conveyance at that location will result in adverse downstream impacts.	Capital Budget / Dedication through development process	Short-Term
Action 4.2 : Acquire sufficient right-of-way to introduce a bench channel section in implementing Alternative 6, and in other areas where feasible.	Capital Budget	Immediate; Ongoing
Action 5.1 : The current level of risk for 100-year and 25-year events should be made freely available through dissemination of floodplain maps, both paper and digital.	CCDD#5 / City of Harlingen / Cameron County	Immediate
Action 5.2 : Make an initial presentaiton to the Chamber of Commerce, and follow up with annual update presentations, or contribute articles to the Chamber's newsletter with updates on CCDD#5 activities.	-	Long-Term; Ongoing
Action 5.3: Identify neighborhood leaders in flood-prone neighborhoods and develop a specific outreach campaign with their guidance.	-	Short-term; On-going
Action 5.4: Work with private inudstry and other stakeholders to develop and implement a program to distribute NOAA All Hazards Weather Radios to the public.	Public-Private Partnership; grant funding	Long-Term
Action 5.5: Working with other authorities, develop a specific plat note requirement to explain the limitations of flood protection in the Lower Rio Grande Valley.	Joint task force of City, District, County, Towns	Short-Term (together with 2.1, 2.2)
Action 6.1 : Recognizing that the illegal dumping problem is a regional issue, work with the Lower Rio Grande Valley Development Council (LRGVDC) to find the best long-term solutions	Multiple regional entities	Short-Term / On-going
Action 6.2 : Pursue grant funded opportunities through TCEQ and LRGVDC to host "clean-up" activities	Capital Budgets; TCEQ grant funding	Short-Term
Action 6.3 : Develop a public awareness program including signs, slogans, posters, etc.	Multiple regional entities	Short-Term
Action 6.4 : Install gates at access points	Capital Budgets	Long-Term
Action 6.5 : Reach out to neighborhood leaders to explain the issue and risks at-hand, and solicit their input on ways to curb the problem and raise awareness	-	Short-term; On-going (together with 5.3)
Action 7.1 : Continue the installation of telemtry-based gages toa monitor flow, stage, and velocity	Capital Budget; FEMA funding w/County and City of Harlingen	Long-Term
Action 7.2 : Perform a model update on a bi-annual basis to incorporate new development and calibration data, if available.	Capital Budget	Long-Term
Action 7.3 : Assess and prioritize the remaining construction projects, knowing that many conditions in the watershed will change over time	-	Long-Term; On-going

8.4 FUNDING SOURCES

An important aspect of implementing any of the recommending alternatives is the funding mechanism. The summary below provides a description of the possible funding sources for the District to construct a project.

8.4.1 Local Entity Funding Sources

Many of these local funding sources are limited to municipalities, as empowered by the State Legislature. The District can cooperatively work with the municipalities in its service area to implement projects which are in part funded by these sources.

Capital Improvements Plan (CIP) - a long-range plan, usually four to six years, which identifies capital projects and equipment purchases, provides a planning schedule and identifies options for financing the plan. The District should prepare a CIP each year during its budget cycle.

Drainage Utility Fees - Municipal stormwater projects are funded by the assessment of a drainage utility fee for all developed projects based on amount of impervious cover, number of living units, or site area.

Development Impact Fees – In accordance with Chapter 395 of Texas Local Government Code, municipalities may impose an impact fee to cover the cost of improvements that are necessitated by new development.

General Fund – The primary operating fund of a governmental entity.

General Obligation Bond (GO) - A municipal bond that is backed by the credit and "taxing power" of the issuing jurisdiction, rather than the revenue from a given project. General obligation bonds are issued with the belief that a municipality will be able to repay its debt obligation through taxation or revenue from projects. No assets are used as collateral. These bonds are typically considered the most secure type of municipal bond, and therefore carry the lowest interest rate.

Revenue Bond - A municipal bond supported by a specified stream of future income, such as income generated by a water utility from payments by customers. This differs from general-obligation bonds, which can be repaid through a variety of tax sources. Revenue bonds are only payable from specified revenues. A main reason for using revenue bonds is that they allow the municipality to avoid reaching legislated debt limits.

Special Assessment Bond - A special type of municipal bond used to fund a development project based on property tax assessments of properties located within the issuer's boundaries.

Tax Increment Bond – A bond (also known as a "tax allocation bond") payable from the incremental increase in tax revenues realized from any increase in property value resulting from capital improvements benefiting the properties that are financed with bond proceeds. Tax increment bonds often are used to finance the redevelopment of blighted areas.

8.4.2 State Funding Assistance Sources

TWDB (Texas Water Development Board)

- Clean Water State Revolving Fund - Provides perpetual funds to provide low interest loan assistance for the planning, design, and construction of stormwater pollution control projects.
- Research and Planning Fund Grants – The purpose is to provide financial assistance for research and feasibility studies into practical solutions to water-related problems.
- State Participation and Storage Acquisition Program – The purpose is to help finance regional water projects including water storage facilities and flood retention basins; and to allow for "right sizing" of projects in consideration of future growth.
- Texas Water Development Fund – The purpose is to provide loans for the planning, design, and construction of water supply, wastewater, and flood control projects.

TCEQ (Texas Commission on Environmental Quality)

- Texas Clean Rivers Program (CRP) – The purpose of these funds are to maintain and improve the quality of surface water resources within each river basin in Texas.

8.4.3 Federal Assistance Sources

FEMA (Federal Emergency Management Agency)

- Flood Hazard Mapping Program – Department of Homeland Security (DHS) funds are administered through FEMA to identify, publish, and update information on all flood-prone areas of the U.S. in order to inform the public on flooding risks, support sound floodplain management, and set flood insurance premium rates.
- Flood Mitigation Assistance Grants (FMA) – The purpose is to assist states and communities in implementing measures to reduce or eliminate the long-term risk of flood damage to buildings, manufactured homes, and other structures insured through the National Flood Insurance Program (NFIP).
- Hazard Mitigation Grant Program (HMGP) – The purpose is to provide states and local governments financial assistance to permanently reduce or eliminate future damages and losses from natural hazards through safer building practices and improving existing structures and supporting infrastructure.
- Pre-Disaster Mitigation Grant Program (PDM) – The purpose is to provide funding for states and communities for cost-effective hazard mitigation activities that complement a comprehensive hazard mitigation program and reduce injuries, loss of life, and dame and destruction of property.

HUD (U.S. Department of Housing and Urban Development)

- Disaster Relief/ Urgent Needs Fund of Texas - To rebuild viable communities impacted by a natural disaster or urgent, unanticipated needs posing serious threats to health and safety by providing decent housing, suitable living environments and economic opportunities.
- Texas Community Development Program – The purpose is to build viable communities that meet "basic human needs" such as safe and sanitary sewer systems, clean drinking water, disaster relief and urgent needs, housing, drainage and flood control, passable streets, and economic development.

NRCS (Natural Resources Conservation Service)

- Watershed Protection and Flood Prevention Program – To protect, develop, and utilize the land and water resources in small watersheds of 250,000 acres or less. The program is Federally assisted and locally led.
- Watershed Surveys and Planning – Provides planning assistance to Federal, State, and local agencies for the development of coordinated water and related land resources programs in watersheds and river basins. Emphasis on flood damage reduction, erosion control, water conservation, preservation of wetlands, and water quality improvements.
- Wetlands Reserve Program – To protect and restore wetlands by enabling landowners to sell easements which take wetlands out of production.
- Emergency Watershed Protection Program – The purpose is to provide relief from imminent hazards and reduce the threat to life and property in the watersheds damaged by severe natural events. Hazards include floods and the products of erosion created by floods, fire, windstorms, earthquakes, drought, or other natural disasters.

USACE (United States Army Corps of Engineers)

- Emergency Advance Measures for Flood Prevention – The purpose is to protect against the loss of life or damages to property given an immediate threat of unusual flooding.
- Emergency Rehabilitation of Flood Control Works – The purpose of this program is to assist in the repair or restoration of flood control works damaged by flood.
- Emergency Streambank and Shoreline Protection – The purpose is to prevent erosion damages to public facilities by the emergency construction or repair of streambank and shoreline protection works.
- Floodplain Management Services – The purpose is to promote appropriate recognition of flood hazards in land and water use planning and development through the provision of flood and floodplain related data, technical services, and guidance.
- Nonstructural Alternatives to Structural Rehabilitation of Damaged Flood Control Works – This program provides a nonstructural alternative to the structural rehabilitation of flood control works damaged in floods or coastal storms.
- Planning Assistance to States – The purpose is to assist states, local governments and other non-Federal entities in the preparation of comprehensive plans for the development, utilization, and conservation of water and related land resources.
- Small Flood Control Projects – The purpose is to reduce flood damages through small flood control projects not specifically authorized by Congress.

8.5 REGULATORY COMPLIANCE

Prior to commencement of construction, it will be necessary to submit the project and appropriate permit applications to regulatory agencies. A detailed review and acquisition of the necessary permits for the construction of these project(s) exceeds the scope of this contract. However, a partial list and brief discussion of permits is included in the following subsections. This following list of agencies and corresponding permit activities is intended to be general in nature and is not intended to represent a definitive list of required permit acquisitions and agency coordination.

8.5.1 Federal Emergency Management Agency (FEMA)

The National Flood Insurance Act of 1968 was enacted by Title XIII of the Housing and Urban Development Act of 1968 (Public Law 90-448, August 1, 1968) to provide previously unavailable flood insurance protection to property owners in flood prone areas. FEMA administers the National Flood Insurance Program (NFIP); however, if a local community elects to participate in the NFIP, the local government is primarily responsible for enforcement. Participating communities are typically covered by FIS which define water surface profiles and floodplain boundaries through their communities.

The recommended drainage improvement projects summarized in this report are intended to reduce floodplain limits. However, if changes to the current effective FEMA floodplain elevations are desirable based on the results of this study, or from the proposed improvements, a request for a Letter of Map Revision (LOMR) from FEMA will be required.

8.5.2 U. S. Army Corps of Engineers (USACE)

Pursuant to Section 404 of the Clean Water Act and the Rules and Regulations promulgated there under by the United States Environmental Protection Agency (USEPA) and the United States Army Corps of Engineers (USACE), the filling or excavation of waters of the United States, including wetlands, with dredged or fill material, requires the issuance of a permit from the USACE (33 CFR Parts 320-330). For purposes of administering the Section 404 permit program, the USACE defines wetlands as follows:

> *Those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas. (33 CFR 328.3)*

The *Corps of Engineers Wetlands Delineation Manual (Technical Report Y-87-1)*, issued by the USACE in 1987 states that wetlands must possess three essential characteristics. These characteristics include, under normal circumstances: 1) the presence of hydrophytic vegetation, 2) hydric soils, and 3) wetland hydrology. If all three of these criteria are present on a particular property in areas larger than one-third acre in size, then a permit (general permit or nationwide permit) must be issued by the USACE in order to fill all or a portion of those areas.

Section 404 (b)(1) guidelines (40 CFR Part 230), established by the USEPA, constitute the substantive environmental criteria used in the evaluating activities regulated under Section 404 of the Clear Water Act. The purpose of these guidelines is to restore and maintain the chemical physical and biological integrity of waters of the United States through the control of discharge of dredged or fill material.

All property owners within the United States and its territories must adhere to the provisions of the Clean Water Act. If any contemplated activity might impact waters of the United States, including adjacent or isolated wetlands a permit application must be made. If jurisdictional waters and/or wetlands are found to exist, then any activity which would involve filling, excavating, or dredging these wetlands would require the issuance of a permit. The final authority to determine whether or not jurisdictional waters exist lies with USACE.

8.5.3 U.S. Fish and Wildlife Service (USFWS)

The U.S. Fish and Wildlife Service (USFWS), in the Department of the Interior, and the National Marine Fisheries Service (NMFS), in the Department of Commerce, share responsibility for administration of the Endangered Species Act (ESA). Generally, the USFWS is responsible for terrestrial and freshwater species and migratory birds, while the NMFS deals with those species occurring in marine environments and anadromous fish.

Section 9 of the ESA prohibits take of federally listed endangered or threatened species without appropriate authorization. Take is defined in the ESA, in part as "killing, harming, or harassment" of a federally listed species, while incidental take is take that is "incidental to, and not the purpose of, otherwise lawful activities".

Section 10 of the ESA provides a means for non-Federal projects resulting in take of listed species to be permitted subject to carefully prescribed conditions. Application for an incidental take permit is subject to a number of requirements, including preparation of a Habitat Conservation Plan by the applicant. In processing an incidental take permit application, the USFWS must comply with appropriate environmental laws, including the National Environmental Policy Act. Review of the application under Section 7 of the ESA is also required to ensure that permit issuance is not likely to jeopardize listed species. Section 10 issuance criteria require the USFWS to issue and incidental take permit if, after opportunity for public comment, it finds that:

1. the taking will be incidental;
2. the applicant will, to the maximum extent practicable, minimize and mitigate the impacts of the taking;
3. the applicant will ensure that adequate funding and means to deal with unforeseen circumstances will be provided;
4. the taking will not appreciably reduce the likelihood of the survival and recovery of the species in the wild; and

5. the applicant will ensure that other measures that the USFWS may require as being necessary or appropriate will be provided.

The U.S. Fish and Wildlife Service should be contacted to determine the potential occurrence of and consequent impacts to any federal threatened and endangered species. In addition, the Corps of Engineers will require USFWS review of the project to ensure the project is in compliance with the Endangered Species Act prior to the issuance of a Section 404 permit.

8.5.4 Texas Commission on Environmental Quality (TCEQ)

The Texas Commission on Environmental Quality (TCEQ) has regulatory authority over: dam safety, the Edwards Aquifer, water rights, Texas Pollutant Discharge Elimination System and Section 404(b)(1) guidelines for specification of disposal sites for dredged or fill material. The following sections briefly describe these regulations.

- Texas Pollutant Discharge Elimination System (TPDES)

On September 14, 1998, the USEPA authorized Texas to implement its Texas Pollutant Discharge Elimination System (TPDES) program. TPDES is the state program to carry out the National Pollutant Discharge Elimination System (NPDES), a federal regulatory program to control discharges of pollutants to surface waters of the United States. The TCEQ administers the program, and a permit is required for any construction activity that disturbs one acre or more.

- Section 401 Water Quality Certification

Any activity requiring authorization under Section 404 of the Clean Water Act will also require a Section 401 water quality certification from the TCEQ. In Texas, these regulations are administered by the TCEQ.

8.5.5 Texas Historical Commission

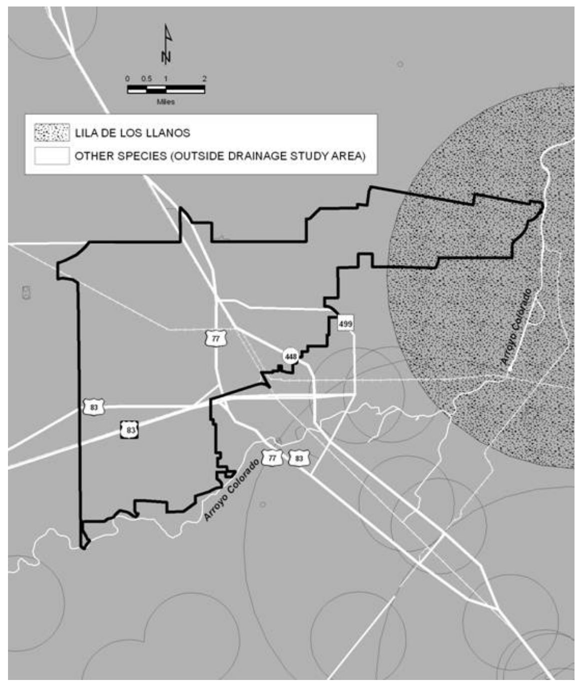
The Division of Antiquities Protection of the Texas Historical Commission coordinates the program by identifying and protecting important archeological and historic sites that may be threatened by public construction projects. This department coordinates the nomination of numerous sites as State Archeological Landmarks or for listing in the *National Register of Historic Places*. Designation is often sought by interested parties as the most effective way to protect archeological sites threatened by new development or vandalism. Applicable rules are found in the Texas Administrative Code, Title 13-Cultural Resources, Part II-Texas Historical Commission, Chapters 24-28.

The Corps of Engineers will require that the State Historical Preservation Officer (SHPO) review the project to ensure the project is in compliance with the National Historic Act prior to issuance of a Section 404 permit.

8.6 ENVIRONMENTAL INVENTORY

The environmental issues of this report have been developed by reference to existing information in published reports, maps, aerial photography, unpublished documents and communications from government agencies, individuals, and private organizations. These issues have been summarized to provide a general review level area studied. Generally, this discussion presents a cursory, screening level perspective on the environmental issues that may affect the study area.

Important species may be considered the local dominant (most abundant) species, species having some economic or recreational importance, those exhibiting disproportionate habitat impacts (habitat formers) as well as species listed, or proposed for listing, by either the State of Texas or the federal government (protected species) or Texas Organization for Endangered Species (TOES). There are numerous unlisted species which are still of concern (due to their rarity, restricted distribution, direct exploitation, or habitat vulnerability), yet have not been included in this discussion. Typically, the level of detail required to obtain the distribution and life history of these species, so as to produce a substantive evaluation, would be beyond the scope of this screening level survey.



The only endangered species identified in the CCDD5 area is the plant, Lila de Los Llanos. This figure below shows the plant to be found near the outfall of North Main Drain. In connection with implementation of Alternative 6, a more detailed environmental analysis should be performed, to determine the presence of this species and an appropriate plan of action.

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APPENDIX A
EXHIBITS

Exhibit 1 – Drainage Area Map
Exhibit 2 – Soils Map
Exhibit 3 – Existing Impervious Cover Map
Exhibit 4 – Future Land Use Map
Exhibit 5 – North Main Drain HEC-RAS Cross-Section Location Maps
Exhibit 6 – Stuart Place Main Drain HEC-RAS Cross-Section Location Maps
Exhibit 7 – Dixieland Main Drain HEC-RAS Cross-Section Location Maps
Exhibit 8 – Southwest Main Drain HEC-RAS Cross-Section Location Maps
Exhibit 9 – 10% and 1% Event North Main Drain Existing Floodplain Maps
Exhibit 10 – 10% and 1% Event Stuart Place Main Drain Existing Floodplain Maps
Exhibit 11 – 10% and 1% Event Dixieland Main Drain Existing Floodplain Maps
Exhibit 12 – 10% and 1% Event Southwest Main Drain Existing Floodplain Maps
Exhibit 13 – Alternative 1 Map
Exhibit 14 – Alternative 2 Map
Exhibit 15 – Alternative 3 Map
Exhibit 16 – Alternative 4 Map
Exhibit 17 – Alternative 5 Map
Exhibit 18 – Alternative 6 Map
Exhibit 19 – Alternative 7 Map
Exhibit 20 – Alternative 8 Map
Exhibit 21 – Alternative 9 Map
Exhibit 22 – Alternative 10 Map
Exhibit 23 – Alternative 1 Water Surface Profile
Exhibit 24 – Alternative 6 Water Surface Profile

APPENDIX B
CURVE NUMBER CALCULATIONS

Basin	Soil Group A	Area (sq ft)			Total Area (sq ft)	Percent of Soil Type				Curve Number		
		Soil Group B	Soil Group C	Soil Group D		% A	% B	% C	% D	AMC I	AMC II	AMC III
DM-01		1,155,765		12,993,266	14,149,031	0%	8%	0%	92%	57.1	75.3	87.4
DM-02		4,759,920		10,152,414	14,912,334	0%	32%	0%	68%	51.7	70.3	84.0
DS-01		16,724,818		17,067,837	33,792,655	0%	49%	0%	51%	47.6	66.6	81.6
NM-01		10,991,644		477,516	11,469,160	0%	96%	0%	4%	37.0	56.9	75.1
NM-02		8,895,889		242,827	9,138,716	0%	97%	0%	3%	36.6	56.6	74.9
NM-03		12,752,330		14,431,379	27,183,709	0%	47%	0%	53%	48.2	67.1	82.0
NM-04		18,180,787		11,235,955	29,416,742	0%	62%	0%	38%	44.8	64.0	79.9
NM-05		10,160,027		10,027,769	20,187,797	0%	50%	0%	50%	47.4	66.4	81.5
NM-06		20,049,869		5,366,247	25,416,116	0%	79%	0%	21%	40.9	60.4	77.5
NM-07		8,527,220		2,544,127	11,071,346	0%	77%	0%	23%	41.3	60.8	77.7
NM-08		17,221,897		27,702,381	44,924,278	0%	38%	0%	62%	50.2	68.9	83.2
NM-09		3,985,428		16,250,911	20,236,339	0%	20%	0%	80%	54.5	72.9	85.8
NM-10		5,713,307		37,991,714	43,705,021	0%	13%	0%	87%	56.0	74.3	86.7
NM-11		1,110,063		7,866,616	8,976,679	0%	12%	0%	88%	56.2	74.4	86.8
NM-12		7,744,106		22,430,301	30,174,408	0%	26%	0%	74%	53.1	71.6	84.9
NM-13		7,401,824		19,635,399	27,037,223	0%	27%	0%	73%	52.7	71.3	84.7
NM-14		5,131,568		15,425,733	20,557,301	0%	25%	0%	75%	53.3	71.8	85.0
NM-15		2,246,044		24,744,602	26,990,646	0%	8%	0%	92%	57.1	75.3	87.3
NM-16		8,543,525		19,430,022	27,973,547	0%	31%	0%	69%	52.0	70.6	84.2
NM-17		6,276,985		13,681,085	19,958,071	0%	31%	0%	69%	51.8	70.4	84.1
NM-18		10,765,340		6,795,830	17,561,170	0%	61%	0%	39%	44.9	64.1	79.9
NM-19		24,637,725	940,154	25,155,982	50,733,861	0%	49%	2%	50%	47.7	66.7	81.6
NM-20		14,722,321	191,513	26,324,540	41,238,374	0%	36%	0%	64%	50.8	69.5	83.5
NM-21		16,358,824		1,379,411	17,738,234	0%	92%	0%	8%	37.8	57.6	75.6
NM-22		13,408,784		31,330,866	44,739,650	0%	30%	0%	70%	52.1	70.7	84.3
NM-23		3,296,354		18,120,974	21,417,328	0%	15%	0%	85%	55.5	73.8	86.4
NM-24		468,235		1,560,202	2,028,438	0%	23%	0%	77%	53.7	72.2	85.3
NM-25		2,485,084		13,006,965	15,492,049	0%	16%	0%	84%	55.3	73.6	86.3
NM-26		9,675,930		19,662,581	29,338,511	0%	33%	0%	67%	51.4	70.1	83.9
OUTSIDE		11,945,497		1,974,829	13,920,325	0%	86%	0%	14%	39.3	59.0	76.5
SP-01		4,000,895		5,161,326	9,162,220	0%	44%	0%	56%	49.0	67.8	82.4
SP-02		6,711,312		9,638,641	16,349,953	0%	41%	0%	59%	49.6	68.4	82.8
SP-03		36,239,125		2,745,088	38,984,213	0%	93%	0%	7%	37.6	57.5	75.5
SP-04		26,194,789		2,225,030	28,419,819	0%	92%	0%	8%	37.8	57.6	75.6
SP-05		19,675,978		12,951,423	32,627,401	0%	60%	0%	40%	45.1	64.3	80.1
SP-06		6,801,182		6,848,045	13,649,227	0%	50%	0%	50%	47.5	66.5	81.5
SP-07		2,406,443		5,552,971	7,959,414	0%	30%	0%	70%	52.0	70.7	84.3
SP-08		17,011,330		2,611,079	19,622,408	0%	87%	0%	13%	39.1	58.8	76.4
SP-09		6,388,040		4,829,423	11,217,463	0%	57%	0%	43%	45.9	65.0	80.6
SP-10		6,670,682		4,739,947	11,410,629	0%	58%	0%	42%	45.6	64.7	80.3
SP-11		2,860,820		12,128,922	14,989,742	0%	19%	0%	81%	54.6	73.0	85.8
SP-12		332,216		6,914,387	7,246,603	0%	5%	0%	95%	57.9	76.0	87.9
SW-01		13,529,071		10,245,894	23,774,965	0%	57%	0%	43%	45.9	65.1	80.6
SW-02		2,561,732		2,908,943	5,470,676	0%	47%	0%	53%	48.2	67.2	82.0
TL-01		16,833,543		2,305,588	19,139,130	0%	88%	0%	12%	38.8	58.5	76.2
WT-01		18,466,210		2,492,963	20,959,173	0%	88%	0%	12%	38.7	58.5	76.2
WT-02		33,335,764		3,897,167	37,232,931	0%	90%	0%	10%	38.4	58.2	76.0
WT-03		14,629,500		605,554	15,235,054	0%	96%	0%	4%	36.9	56.8	75.1
WT-04		10,141,737			10,141,737	0%	100%	0%	0%	36.0	56.0	74.5
WT-05		28,253,243		3,991,586	32,244,830	0%	88%	0%	12%	38.8	58.6	76.3
WT-06		14,713,756		753,886	15,467,642	0%	95%	0%	5%	37.1	57.0	75.2
WT-07		8,121,151		1,583,504	9,704,655	0%	84%	0%	16%	39.8	59.4	76.8
WT-08		7,873,365		635,627	8,508,992	0%	93%	0%	7%	37.7	57.6	75.6
WT-09		3,615,619		1,371,154	4,986,773	0%	73%	0%	27%	42.3	61.8	78.4
WT-10		7,747,172		8,226,269	15,973,441	0%	49%	0%	51%	47.8	66.8	81.7
WT-11		7,819,766		11,039,601	18,859,367	0%	41%	0%	59%	49.5	68.3	82.7
WT-12				3,232,753	3,232,753	0%	0%	0%	100%	59.0	77.0	88.5
WT-13		20,523,918		5,440,725	25,964,643	0%	79%	0%	21%	40.8	60.4	77.5
WT-14		9,715,536		9,741,429	19,456,964	0%	50%	0%	50%	47.5	66.5	81.5
WT-15		7,445,827		8,226,468	15,672,296	0%	48%	0%	52%	48.1	67.0	81.9

Curve Numbers (CN)			
Soil Group	AMC I	AMC II	AMC III
A	19	35	55
B	36	56	75
C	51	70	84
D	59	77	89

Brush cover type under Fair Hydrologic Condition

Source TR-55

APPENDIX C
EXISTING AND ULTIMATE CONDITIONS
IMPERVIOUS COVER PERCENTAGES

Sub-area	Existing % Impervious Cover	Ultimate % Impervious Cover	Sub-area	Existing % Impervious Cover	Ultimate % Impervious Cover
DM-01	11	55	SP-01	9	55
DM-02	14	51	SP-02	28	71
DS-01	4	47	SP-03	16	61
NM-01	5	15	SP-04	23	58
NM-02	16	56	SP-05	20	56
NM-03	5	37	SP-06	5	50
NM-04	23	26	SP-07	7	50
NM-05	24	60	SP-08	7	50
NM-06	18	32	SP-09	6	50
NM-07	5	50	SP-10	8	51
NM-08	33	68	SP-11	4	50
NM-09	22	44	SP-12	3	52
NM-10	43	75	SW-01	10	50
NM-11	36	68	SW-02	4	50
NM-12	13	64	TL-01	10	52
NM-13	38	58	WT-01	4	14
NM-14	28	60	WT-02	3	13
NM-15	25	58	WT-03	7	17
NM-16	8	64	WT-04	2	16
NM-17	3	50	WT-05	4	24
NM-18	4	51	WT-06	5	40
NM-19	3	13	WT-07	13	24
NM-20	3	13	WT-08	18	16
NM-21	3	13	WT-09	5	24
NM-22	9	19	WT-10	7	16
NM-23	4	14	WT-11	10	25
NM-24	2	10	WT-12	29	90
NM-25	3	13	WT-13	9	58
NM-26	1	11	WT-14	38	68
OUTSIDE	15	40	WT-15	3	50

APPENDIX D
TIME OF CONCENTRATION
EXISTING CONDITIONS
KERBY-KIRPICH METHOD

Basin	WT01	WT02	WT03	WT04	WT05	WT06	WT07	WT08	WT09	WT10	WT11	WT12	WT13	WT14
Kerby Overland Flow														
Retardance Coefficient, N	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.1	0.3	0.3	0.3	0.3	0.3	0.3
Length, L (ft)	100	270	300	250	250	250	300	250	250	300	250	150	250	250
Slope, S	0.0037	0.0010	0.0022	0.0085	0.0073	0.0018	0.0012	0.0001	0.0094	0.0033	0.0051	0.0003	0.0001	0.0012
Kerby Sheet Flow Tc (min)	15.1	32.3	28.4	19.0	19.7	27.3	32.7	32.0	18.5	25.8	21.4	31.5	53.5	29.7
Length, L (ft)	900	300	500	1,100	1,200	1,150	1,100	750	1,200	1,200	950	600	500	750
Slope, S	0.001	0.009	0.010	0.002	0.003	0.000	0.001	0.000	0.001	0.001	0.001	0.000	0.001	0.001
Kerby Shallow Concentrated Tc (min)	56.5	20.3	25.2	56.4	50.8	74.9	68.2	53.5	68.5	67.0	61.5	94.7	40.6	50.2
Kirpich Channelized Flow														
Length, L (ft)	1,895	3,430	1,700	664	4,882	2,660	1,250	1,025	680	3,000	5,600	630	1,500	400
Slope, S	0.0013	0.0010	0.0004	0.0015	0.0006	0.0004	0.0009	0.0001	0.0028	0.0001	0.0004	0.0055	0.0010	0.0007
Kirpich Channel 1 Tc (min)	33.5	58.3	49.7	14.3	94.0	67.6	27.7	56.3	11.3	139.4	118.5	8.3	31.5	12.6
Length, L (ft)	1,266	3,302	1,800	1,486	2,206	840	1,176	2,475	1,520	2,000	400	220	2,550	736
Slope, S	0.0010	0.0012	0.0006	0.0012	0.0011	0.0016	0.0011	0.0023	0.0004	0.0022	0.0080	0.0187	0.0010	0.0017
Kirpich Channel 2 Tc (min)	27.0	52.9	43.5	29.1	40.8	16.8	24.8	33.2	45.4	28.8	5.0	2.3	46.5	14.7
Length, L (ft)	2,730	2,486	1,707	940	3,695	2,011	1,030	237	685	722	3,087	1,637	5,485	5,403
Slope, S	0.0010	0.0009	0.0024	0.0014	0.0005	0.0001	0.0023	0.0008	0.0015	0.0015	0.0011	0.0004	0.0007	0.0001
Kirpich Channel 3 Tc (min)	50.1	47.2	24.4	19.2	84.3	89.6	16.9	8.2	14.4	15.3	51.7	47.5	98.1	185.1
Kerby-Kirpich, Total Tc (min)	**182.2**	**210.9**	**171.3**	**138.0**	**289.6**	**276.3**	**170.3**	**183.2**	**158.1**	**276.2**	**258.1**	**184.3**	**270.3**	**292.3**

Basin	NM15	NM16	NM17	NM18	NM19	NM20	NM21	NM22	NM23	NM24	NM25	NM26	SP01	SP02
Kerby Overland Flow														
Retardance Coefficient, N	0.1	0.1	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Length, L (ft)	250	250	250	250	250	250	250	400	500	250	250	300	300	250
Slope, S	0.0001	0.0001	0.0004	0.0001	0.0034	0.0007	0.0001	0.0001	0.0001	0.0001	0.0001	0.0022	0.0072	0.0056
Kerby Sheet Flow Tc (min)	32.0	32.0	39.8	48.9	23.4	34.3	53.5	66.6	74.0	53.5	53.5	28.4	21.5	20.9
Length, L (ft)	750	700	750	750	750	800	1,700	1,000	800	750	2,400	1,000	1,200	1,300
Slope, S	0.000	0.000	0.000	0.000	0.001	0.000	0.000	0.001	0.002	0.000	0.000	0.001	0.001	0.000
Kerby Shallow Concentrated Tc (min)	53.5	51.8	80.0	89.4	54.4	92.1	108.3	62.1	46.0	89.4	153.9	55.9	72.3	118.5
Kirpich Channelized Flow														
Length, L (ft)	500	3,682	3,024	650	4,608	3,286	2,436	5,278	1,100	650	270	2,100	2,200	919
Slope, S	0.0001	0.0000	0.0005	0.0055	0.0004	0.0005	0.0003	0.0001	0.0013	0.0016	0.0224	0.0007	0.0003	0.0026
Kirpich Channel 1 Tc (min)	32.4	220.2	67.9	8.5	109.0	72.7	69.3	198.8	22.4	13.8	2.5	45.1	64.3	14.8
Length, L (ft)	1,700	332	736	850	502	2,464	127	160	1,867	1,085	680	3,074	2,800	2,863
Slope, S	0.0001	0.0269	0.0013	0.0019	0.0066	0.0005	0.0518	0.0790	0.0003	0.0006	0.0051	0.0018	0.0008	0.0010
Kirpich Channel 2 Tc (min)	83.1	2.7	16.3	15.8	6.5	58.1	1.0	1.0	56.1	30.0	9.0	43.3	55.0	51.1
Length, L (ft)	2,858	2,765	1,215	3,516	7,058	3,492	425	2,639	815	149	2,734	2,663	1,520	3,663
Slope, S	0.0020	0.0019	0.0087	0.0025	0.0008	0.0007	0.0018	0.0004	0.0029	0.0575	0.0027	0.0061	0.0027	0.0020
Kirpich Channel 3 Tc (min)	39.1	39.0	11.5	42.0	109.6	70.1	9.3	68.3	12.8	1.1	33.5	24.1	21.5	47.7
Kerby-Kirpich, Total Tc (min)	**240.2**	**345.8**	**215.5**	**204.6**	**302.9**	**327.2**	**241.5**	**396.8**	**211.4**	**187.7**	**252.4**	**196.9**	**234.5**	**252.9**

Basin	WT01	WT02	WT03	WT04	WT05	WT06	WT07	WT08	WT09	WT10	WT11	WT12	WT13	WT14
Kerby Overland Flow														
Retardance Coefficient, N	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Length, L (ft)	100	270	300	250	250	250	300	250	250	300	250	150	250	250
Slope, S	0.0037	0.0010	0.0022	0.0085	0.0073	0.0018	0.0012	0.0001	0.0094	0.0033	0.0051	0.0003	0.0001	0.0012
Kerby Sheet Flow Tc (min)	9.0	19.3	17.0	11.4	11.8	16.3	19.5	32.0	11.1	15.4	12.8	18.8	32.0	17.8
Length, L (ft)	900	300	500	1,100	1,200	1,150	1,100	750	1,200	1,200	950	600	500	750
Slope, S	0.001	0.009	0.010	0.002	0.003	0.000	0.001	0.000	0.001	0.001	0.001	0.000	0.001	0.001
Kerby Shallow Concentrated Tc (min)	33.8	12.1	15.1	33.8	30.4	44.9	40.8	53.5	41.0	40.1	36.8	56.7	24.3	30.0
Kirpich Channelized Flow														
Length, L (ft)	1,895	3,430	1,700	664	4,882	2,660	1,250	1,025	680	3,000	5,600	630	1,500	400
Slope, S	0.0013	0.0010	0.0004	0.0015	0.0006	0.0004	0.0009	0.0001	0.0028	0.0001	0.0004	0.0055	0.0010	0.0007
Kirpich Channel 1 Tc (min)	33.5	58.3	49.7	14.3	94.0	67.6	27.7	56.3	11.3	139.4	118.5	8.3	31.5	12.6
Length, L (ft)	1,266	3,302	1,800	1,486	2,206	840	1,176	2,475	1,520	2,000	400	220	2,550	736
Slope, S	0.0010	0.0012	0.0006	0.0012	0.0011	0.0016	0.0011	0.0023	0.0004	0.0022	0.0080	0.0187	0.0010	0.0017
Kirpich Channel 2 Tc (min)	27.0	52.9	43.5	29.1	40.8	16.8	24.8	33.2	45.4	28.8	5.0	2.3	46.5	14.7
Length, L (ft)	2,730	2,486	1,707	940	3,695	2,011	1,030	237	685	722	3,087	1,637	5,485	5,403
Slope, S	0.0010	0.0009	0.0024	0.0014	0.0005	0.0001	0.0023	0.0008	0.0015	0.0015	0.0011	0.0004	0.0007	0.0001
Kirpich Channel 3 Tc (min)	50.1	47.2	24.4	19.2	84.3	89.6	16.9	8.2	14.4	15.3	51.7	47.5	98.1	185.1
Kerby-Kirpich, Total Tc (min)	**153.5**	**189.8**	**149.7**	**107.7**	**261.3**	**235.3**	**129.8**	**183.2**	**123.2**	**239.0**	**224.8**	**133.6**	**232.5**	**260.2**

Basin	NM15	NM16	NM17	NM18	NM19	NM20	NM21	NM22	NM23	NM24	NM25	NM26	SP01	SP02
Kerby Overland Flow														
Retardance Coefficient, N	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Length, L (ft)	250	250	250	250	250	250	250	400	500	250	250	300	300	250
Slope, S	0.0001	0.0001	0.0004	0.0001	0.0034	0.0007	0.0001	0.0001	0.0001	0.0001	0.0001	0.0022	0.0072	0.0056
Kerby Sheet Flow Tc (min)	32.0	32.0	23.8	29.3	14.0	20.5	32.0	39.9	44.3	32.0	32.0	17.0	12.8	12.5
Length, L (ft)	750	700	750	750	750	800	1,700	1,000	800	750	2,400	1,000	1,200	1,300
Slope, S	0.000	0.000	0.000	0.000	0.001	0.000	0.000	0.001	0.002	0.000	0.000	0.001	0.001	0.000
Kerby Shallow Concentrated Tc (min)	53.5	51.8	47.9	53.5	32.6	55.1	64.8	37.1	27.6	53.5	92.1	33.5	43.3	71.0
Kirpich Channelized Flow														
Length, L (ft)	500	3,682	3,024	650	4,608	3,286	2,436	5,278	1,100	650	270	2,100	2,200	919
Slope, S	0.0001	0.0000	0.0005	0.0055	0.0004	0.0005	0.0003	0.0001	0.0013	0.0016	0.0224	0.0007	0.0003	0.0026
Kirpich Channel 1 Tc (min)	32.4	220.2	67.9	8.5	109.0	72.7	69.3	198.8	22.4	13.8	2.5	45.1	64.3	14.8
Length, L (ft)	1,700	332	736	850	502	2,464	127	160	1,867	1,085	680	3,074	2,800	2,863
Slope, S	0.0001	0.0269	0.0013	0.0019	0.0066	0.0005	0.0518	0.0790	0.0003	0.0006	0.0051	0.0018	0.0008	0.0010
Kirpich Channel 2 Tc (min)	83.1	2.7	16.3	15.8	6.5	58.1	1.0	1.0	56.1	30.0	9.0	43.3	55.0	51.1
Length, L (ft)	2,858	2,765	1,215	3,516	7,058	3,492	425	2,639	815	149	2,734	2,663	1,520	3,663
Slope, S	0.0020	0.0019	0.0087	0.0025	0.0008	0.0007	0.0018	0.0004	0.0029	0.0575	0.0027	0.0061	0.0027	0.0020
Kirpich Channel 3 Tc (min)	39.1	39.0	11.5	42.0	109.6	70.1	9.3	68.3	12.8	1.1	33.5	24.1	21.5	47.7
Kerby-Kirpich, Total Tc (min)	**240.2**	**345.8**	**167.4**	**149.1**	**271.7**	**276.5**	**176.6**	**345.1**	**163.2**	**130.4**	**169.2**	**163.0**	**196.9**	**196.9**

APPENDIX E
MAY 25, 2007 PUMP HOUSE RAINFALL DATA

Pump House 18 (PH 18)		
Date & Time	Daily Rain (in)	Incremental (in)
5/25/2007 0:00	0	0
5/25/2007 0:15		
5/25/2007 0:30		
5/25/2007 0:45		
5/25/2007 1:00	0	0
5/25/2007 1:15	0.04	0.04
5/25/2007 1:30		
5/25/2007 1:45		
5/25/2007 2:00		
5/25/2007 2:15		
5/25/2007 2:30		
5/25/2007 2:45		
5/25/2007 3:00		
5/25/2007 3:15	0.04	0
5/25/2007 3:30	0.07	0.03
5/25/2007 3:45	0.08	0.01
5/25/2007 4:00		
5/25/2007 4:15		
5/25/2007 4:30		
5/25/2007 4:45		
5/25/2007 5:00		
5/25/2007 5:15		
5/25/2007 5:30		
5/25/2007 5:45		
5/25/2007 6:00		
5/25/2007 6:15		
5/25/2007 6:30		
5/25/2007 6:45		
5/25/2007 7:00		
5/25/2007 7:15		
5/25/2007 7:30		
5/25/2007 7:45		
5/25/2007 8:00		
5/25/2007 8:15		
5/25/2007 8:30		
5/25/2007 8:45		
5/25/2007 9:00		
5/25/2007 9:15		
5/25/2007 9:30		
5/25/2007 9:45		
5/25/2007 10:00		
5/25/2007 10:15		
5/25/2007 10:30		
5/25/2007 10:45		
5/25/2007 11:00	0.08	0
5/25/2007 11:15	0.11	0.03
5/25/2007 11:30		
5/25/2007 11:45		
5/25/2007 12:00		
5/25/2007 12:15		
5/25/2007 12:30	0.11	0
5/25/2007 12:45	0.12	0.01
5/25/2007 13:00		
5/25/2007 13:15		
5/25/2007 13:30		
5/25/2007 13:45		
5/25/2007 14:00	0.12	0
5/25/2007 14:15	0.21	0.09
5/25/2007 14:30	0.39	0.18
5/25/2007 14:45		
5/25/2007 15:00	0.39	0
5/25/2007 15:15	0.43	0.04
5/25/2007 15:30	1.22	0.79
5/25/2007 15:45	1.91	0.69
5/25/2007 16:00	2.61	0.7
5/25/2007 16:15	2.78	0.17
5/25/2007 16:30	2.83	0.05
5/25/2007 16:45	2.85	0.02
5/25/2007 17:00	2.85	0
5/25/2007 17:15		
5/25/2007 17:30		
5/25/2007 17:45		
5/25/2007 18:00	2.87	0.02
5/25/2007 18:15		
5/25/2007 18:30		
5/25/2007 18:45		
5/25/2007 19:00		
5/25/2007 19:15		
5/25/2007 19:30		
5/25/2007 19:45		
5/25/2007 20:00		
5/25/2007 20:15		
5/25/2007 20:30		
5/25/2007 20:45		
5/25/2007 21:00		
5/25/2007 21:15		
5/25/2007 21:30		
5/25/2007 21:45		
5/25/2007 22:00		
5/25/2007 22:15		
5/25/2007 22:30		
5/25/2007 22:45		
5/25/2007 23:00		
5/25/2007 23:15		
5/25/2007 23:30		
5/25/2007 23:45		
5/26/2007 0:00		

Pump House 42 (PH 42)		
Date & Time	Daily Rain (in)	Incremental (in)
5/25/2007 0:00		0
5/25/2007 0:15	0	
5/25/2007 0:30	0	0
5/25/2007 0:45	0.12	0.12
5/25/2007 1:00	0.18	0.06
5/25/2007 1:15	0.2	0.02
5/25/2007 1:30	0.2	0
5/25/2007 1:45	0.21	0.01
5/25/2007 3:15	0.21	0
5/25/2007 3:30	0.22	0.01
5/25/2007 3:45	0.25	0.03
5/25/2007 10:45	0.25	0
5/25/2007 11:00	0.27	0.02
5/25/2007 12:15	0.27	0
5/25/2007 12:30	0.4	0.13
5/25/2007 12:45	0.48	0.08
5/25/2007 13:15	0.48	0
5/25/2007 13:30	0.49	0.01
5/25/2007 13:45	0.5	0.01
5/25/2007 14:00	0.54	0.04
5/25/2007 14:15	0.95	0.41
5/25/2007 14:30	2.05	1.1
5/25/2007 14:45	2.25	0.2
5/25/2007 15:00	2.43	0.18
5/25/2007 15:15	2.54	0.11
5/25/2007 15:30	2.68	0.14
5/25/2007 15:45	2.84	0.16
5/25/2007 16:00	2.89	0.05
5/25/2007 16:15	2.92	0.03
5/25/2007 16:30	2.95	0.03
5/25/2007 16:45	2.95	0
5/25/2007 17:45	3	0.05
5/26/2007 0:00	3	0

Pump House 55 (PH 55)		
Date & Time	Daily Rain (in)	Incremental (in)
5/25/2007 0:00		0.02
5/25/2007 0:15	0.07	0.04
5/25/2007 0:30	0.21	0.14
5/25/2007 0:45	0.3	0.09
5/25/2007 1:00	0.4	0.1
5/25/2007 1:15	0.41	0.01
5/25/2007 1:30	0.42	0.01
5/25/2007 1:45	0.45	0.03
5/25/2007 2:00	0.49	0.04
5/25/2007 2:15	0.56	0.07
5/25/2007 2:30	0.57	0.01
5/25/2007 2:45	0.66	0.09
5/25/2007 3:00	0.82	0.16
5/25/2007 4:15	0.82	0
5/25/2007 4:30	0.83	0.01
5/25/2007 5:00	0.87	0.04
5/25/2007 10:15	0.87	0
5/25/2007 10:45	1.2	0.33
5/25/2007 11:00	1.3	0.1
5/25/2007 11:45	1.3	0
5/25/2007 12:00	1.34	0.04
5/25/2007 12:15	1.35	0.01
5/25/2007 12:30	1.37	0.02
5/25/2007 12:45	1.53	0.16
5/25/2007 13:00	1.58	0.05
5/25/2007 13:15	2.18	0.6
5/25/2007 13:30	2.44	0.26
5/25/2007 13:45	2.56	0.12
5/25/2007 14:00	2.67	0.11
5/25/2007 14:15	2.7	0.03
5/25/2007 14:30	2.72	0.02
5/25/2007 14:45	2.76	0.04
5/25/2007 15:00	2.99	0.23
5/25/2007 15:15	3.08	0.09
5/25/2007 15:30	3.48	0.4
5/25/2007 15:45	4.25	0.77
5/25/2007 16:00	4.7	0.45
5/25/2007 16:15	4.91	0.21
5/25/2007 16:30	4.93	0.02
5/25/2007 16:45	4.94	0.01
5/25/2007 17:45	4.99	0.05
5/25/2007 18:00	5	0.01
5/25/2007 18:15	5.03	0.03
5/25/2007 18:30	5.05	0.02
5/25/2007 18:45	5.08	0.03
5/26/2007 0:00	5.08	0

APPENDIX F
HEC-RAS OUTPUT REPORTS

APPENDIX G
HEC-HMS OUTPUT REPORT

APPENDIX H
COST ESTIMATE CALCULATIONS

ALTERNATIVE 1

Wilson Tract Diversion

Item	Item Description	Qty	Unit	Unit Cost	Total Amount
Toscano ROW		1	LS		$25,000
Kilbourn Road Crossing		1	LS		$130,000
Cragon Road Crossing		1	LS		$130,000
Expwy 77 Crossing		1	LS		$750,000
Construction Breedlove to Crossett		1	LS		$200,000
Construction Crossett to Expwy 77		1	LS		$100,000
Business 77 and Railroad Crossing		1	LS		$700,000
Kilbourn Road Crossing		1	LS		$130,000
Construction Expwy 77 to Bus 77		1	LS		$75,000
Hand Road Crossing		1	LS		$130,000
Construction Rio Rancho to Bus 77		1	LS		$75,000
ROW Young		1	LS		$112,000
ROW Francis		1	LS		$60,000
Construction Cragon to Rio Rancho		1	LS		$250,000
ROW Cragon to Burns		1	LS		$250,000
Construction w/ crossings Cragon to Burns		1	LS		$700,000
				TOTAL ESTIMATED COST	**$3,817,000**

Note: Costs provided by CCDD5

ALTERNATIVE 2

Railroad Bridge Replacement on North Main

Item	Item Description	Qty	Unit	Unit Cost	Total Amount
Demolition					
	Demolish/Remove Existing Piers/Bridge	900	cy	$ 30	$ 27,000
	On-Site Grading and Berming	900	cy	$ 3	$ 2,700
	Subgrade Prep/Final Grading	900	cy	$ 1	$ 900
RR Replacement					
	Install New Bridge/ Deck/ RR	900	cy	$ 400	$ 360,000
Culvert					
	2-10' by 10' RCB @ 25 LF	50	lf	$ 1,000	$ 50,000
Erosion/Sedimentation Controls					
	Revegetation	400	sy	$ 3.50	$ 1,400
	SWPPP Compliance	1	LS	$ 5,000	$ 5,000
Contingency (20%)					$ 89,400
				SUBTOTAL	**$ 536,400**
Mobilization/ Demobilization (5%)					$ 26,820
Engineering and surveying (20%)					$ 107,280
Project Management (2%)					$ 10,728
Construction Inspection (4%)					$ 21,456
				TOTAL ESTIMATED COST	**$ 702,684**

Note: Does not include relocation of any utilities

ALTERNATIVE 3

Offline Detention Basin near FM 2994 with Additional Excavation

Item	Item Description	Qty	Unit	Unit Cost	Total Amount
Excavation/Fill					
	Excavation Site	90,879	cy	$ 2.00	$ 181,758
Outlet Structure					
	Pipe (4-24" Diameter RCP)	100	lf	$ 100.00	$ 10,000
	Concrete Wier/Embankment	200	cy	$ 150.00	$ 30,000
Landscaping					
	Maintence Road/Hike & Bike Trail	3,600	lf	$ 10.00	$ 36,000
	Pond Access	3,600	lf	$ 10.00	$ 36,000
Erosion/Sedimentation Controls					
	Revegetation	42,113	sy	$ 3.50	$ 147,396
	SWPPP Compliance	1	LS	$ 7,000	$ 7,000
Contingency (20%)					$ 89,631
				SUBTOTAL	**$ 537,785**
Mobilization/ Demobilization (5%)					$ 26,889
Engineering and surveying (20%)					$ 107,557
Project Management (2%)					$ 10,756
Construction Inspection (4%)					$ 21,511
				TOTAL ESTIMATED COST	**$ 704,498**

Note: Does not include relocation of any utilities

ALTERNATIVE 4

Culvert Improvement at Chester Park Irrigation Canal

Item	Item Description	Qty	Unit	Unit Cost	Total Amount
Demolition					
	Demolish/ Reform Irrigation Canal	1	ls	$ 20,000	$ 20,000
	Demolish/Remove Existing Culverts	370	cy	$ 30	$ 11,111
	On-Site Grading and Berming	370	cy	$ 3	$ 1,111
	Subgrade Prep/Final Grading	370	cy	$ 1	$ 370
Culvert					
	1-10' by 10' RCB @ 72 LF	72	lf	$ 800	$ 57,600
Erosion/Sedimentation Controls					
	Revegetation	222	sy	$ 3.50	$ 778
	SWPPP Compliance	1	LS	$ 7,000	$ 7,000
Contingency (20%)					$ 19,594
				SUBTOTAL	**$ 117,564**
Mobilization/ Demobilization (5%)					$ 5,878
Engineering and Surveying (20%)					$ 23,513
Project Management (2%)					$ 2,351
Construction Inspection (4%)					$ 4,703
				TOTAL ESTIMATED COST	**$ 154,009**

Note: Does not include relocation of any utilities

ALTERNATIVE 4A

Culvert Improvement at Chester Park Irrigation Canal with Pump Station Relocation

Item	Item Description	Qty	Unit	Total Unit Cost	Total Amount
Demolition					
	Demolish/Remove Existing Culverts	370	cy	$ 30	$ 11,111
	On-Site Grading and Berming	370	cy	$ 3	$ 1,111
	Subgrade Prep/Final Grading	370	cy	$ 1	$ 370
Irrigation Canal					
	Pump Station Relocation	1	ls	$ 250,000	$ 250,000
Erosion/Sedimentation Controls					
	Revegetation	444	sy	$ 3.50	$ 1,556
	SWPPP Compliance	1	LS	$ 7,000	$ 7,000
Contingency (20%)					$ 54,230
				SUBTOTAL	**$ 325,378**
Mobilization/ Demobilization (5%)					$ 16,269
Engineering and surveying (20%)					$ 65,076
Project Management (2%)					$ 6,508
Construction Inspection (4%)					$ 13,015
				TOTAL ESTIMATED COST	**$ 426,245**

Note: Does not include relocation of any utilities

ALTERNATIVE 5

North Main Full Connection to Stuart Place Main at Acacia

Item	Item Description	Qty	Unit	Total Unit Cost	Total Amount
Weir/Diversion Structure					
	Concrete Wier/Embankment	200	cy	$ 150.00	$ 30,000
Erosion/Sedimentation Controls					
	Revegetation	889	sy	$ 3.50	$ 3,111
	SWPPP Compliance	1	LS	$ 5,000	$ 5,000
Contingency (20%)					$ 7,622
				SUBTOTAL	**$ 37,622**
Mobilization/ Demobilization (5%)					$ 1,881
Engineering and surveying (20%)					$ 7,524
Project Management (2%)					$ 752
Construction Inspection (4%)					$ 1,505
				TOTAL ESTIMATED COST	**$ 49,285**

Note: Does not include relocation of any utilities

ALTERNATIVE 6

North Main Channel Improvements Downstream of US-77

Item	Item Description	Qty	Unit	Unit Cost	Total Amount
Excavation/Fill					
	Channel Excavation	33104	cy	$ 2	$ 66,208
Right of Way					
	Additional Right-of-Way	39	ac	$ 20,000	$ 780,000
	Prep Right-of-Way	1	ls	$ 50,000	$ 50,000
Bridge Improvements					
	FM 507 Replacement	1	ls	$ 500,000	$ 500,000
	FM 508 Adjustment, extend bridge 20'	1	ls	$ 100,000	$ 100,000
	Briggs Coleman Adjustment, extend bridge 20'	1	ls	$ 100,000	$ 100,000
	Briggs Road Adjustment, extend bridge 27'	1	ls	$ 100,000	$ 100,000
	Breedlove Adjustment, extend bridge 37'	1	ls	$ 100,000	$ 100,000
Erosion/Sedimentation Controls					
	Revegetation	352,000	sy	$ 1.75	$ 616,000
	SWPPP Compliance	1	LS	$ 12,000	$ 12,000
Contingency (20%)					$ 359,242
				SUBTOTAL	**$ 2,783,450**
Mobilization/ Demobilization (5%)					$ 139,172
Engineering and surveying (20%)					$ 556,690
Project Management (2%)					$ 55,669
Construction Inspection (4%)					$ 111,338
				TOTAL ESTIMATED COST	**$ 3,646,319**

Note: Does not include relocation of any utilities

ALTERNATIVE 8

Harlingen Country Club Golf Course Channelization

Item	Item Description	Qty	Unit	Total Unit Cost	Total Amount
Excavation/Fill					
	Channel Excavation	12466	cy	$ 2	$ 24,932
	Removal of Culverts	3326	lf	$ 40	$ 133,040
Right of Way					
	Additional Right-of-Way	3	ac	$ 20,000	$ 60,000
	Prep Right-of-Way	1	ls	$ 15,000	$ 15,000
Erosion/Sedimentation Controls					
	Revegetation	37,400	sy	$ 3.5	$ 130,900
	SWPPP Compliance	1	LS	$ 12,000	$ 12,000
Contingency (20%)					$ 75,174
				SUBTOTAL	**$ 451,046**
Mobilization/ Demobilization (5%)					$ 22,552
Engineering and surveying (20%)					$ 90,209
Project Management (2%)					$ 9,021
Construction Inspection (4%)					$ 18,042
				TOTAL ESTIMATED COST	**$ 590,871**

Note: Does not include relocation of any utilities

ALTERNATIVE 10

Primera Improvements

				Total	
Item	**Item Description**	**Qty**	**Unit**	**Unit Cost**	**Amount**
Excavation/Fill					
	Channel Excavation	608	cy	$ 2	$ 1,216
	Channel Fill	704	cy	$ 2	$ 1,408
Culvert					
	2- 5' by 5' Box Culverts @ 50 LF	100	lf	$ 500	$ 50,000
	Wilson Tract Main at Alanzo Road				
	2- 5' by 5' Box Culverts @ 41 LF	82	lf	$ 500	$ 41,000
	Wilson Tract Main at Stuart Place Road				
	2- 5' by 5' Box Culverts @ 27 LF	52	lf	$ 500	$ 26,000
	Wilson Tract Main at Wilcox Road				
	2- 4' by 4' Box Culvert @ 52 LF	104	lf	$ 400	$ 41,600
	South Fork Lateral at Primera Road				
	2- 4' by 4' Box Culvert @ 40 LF	80	lf	$ 400	$ 32,000
	South Fork Lateral near Carver Road				
	2- 4' by 4' Box Culvert @ 44 LF	88	lf	$ 400	$ 35,200
	South Fork Lateral at Railroad				
Erosion/Sedimentation Controls					
	Revegetation	28,418	sy	$ 3.50	$ 99,462
	SWPPP Compliance	1	LS	$ 12,000	$ 12,000
Contingency (20%)					$ 67,977
				SUBTOTAL	**$ 407,863**
Mobilization/ Demobilization (5%)					$ 20,393
Engineering and surveying (20%)					$ 81,573
Project Management (2%)					$ 8,157
Construction Inspection (4%)					$ 16,315
				TOTAL ESTIMATED COST	**$ 534,301**

Note: Does not include relocation of any utilities

APPENDIX I
ALTERNATIVE BENEFIT CALCULATIONS

APPENDIX J
DIGITAL DATA

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Austin, Texas 78704
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