City of

DEL RIO

Master Watershed Study
Final Report
Volume 1

SEPTEMBER 2012

CP&Y

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1.0 INTRODUCTION

1.1 Background

Drainage problems in Del Rio, as with any city, are a recurring issue that represents an ongoing threat to the citizens and a constant burden on the city administration, which must deal with the difficult choices posed by public safety responsibilities, property damage prevention or mitigation, expenditures of limited public funds for infrastructure improvements and an ever-increasing overlay of regulatory and environmental constraints. Del Rio, while located in a semi-arid area (or, as some would argue, a very arid area based on recent drought conditions), has a history of flooding problems that include the entire spectrum of drainage issues ranging from major creek flooding (such as San Felipe Creek) to very localized nuisance flooding in neighborhoods caused by such factors as inadequate surface drainage capacity, lack of storm drainage infrastructure, flat topography, lack of positive overflow and problems associated with individual lots, such as grading and finished floor elevations. Although Del Rio is a medium-sized and growing city, it shares many, if not most, of the drainage problems that are typically experienced by much larger municipalities, as well as some unique issues such as the local canal network and the existence of environmentally-sensitive areas such as San Felipe Springs.

1.2 Authorization

In November 2009, the City of Del Rio (City) requested qualifications for a City Wide Master Drainage Study. In November 2010, the City awarded the contract to CP&Y, Inc. (CP&Y). The original scope of work included eight overall tasks:

1. Perform an inventory of existing drainage ways within the City Limits;
2. Plot the actual drainage areas onto a map;
3. Analyze the flows for the 5-, 25-, and 100-year rainfall events;
4. List the most critical drainage ways that are threatening existing properties from runoff;
5. Prioritize the list so a Capital Improvement Plan may be created to fund and construct improvements;
6. Provide recommendations for the most appropriate type of project to solve the existing problems;
7. Provide an engineer's estimate of approximate cost for each of the projects;
8. Include BMPs that will serve to improve the quality of runoff that reaches San Felipe Creek.

CP&Y expanded the original scope of work with more detail as to the actual creeks and watercourses to be studied. Analysis of historical precipitation and storm events was included. The breadth of the Master Drainage Study includes the development of new hydrologic and hydraulic models from public domain topography combined with on the ground survey of channel geometry and drainage structures. With the new tools of HEC-HMS and HEC-RAS models, CP&Y could quantify the flood impacts at critical road crossings and areas of flooding. Proposed improvements can be compared against existing conditions to determine potential benefits of different alternatives and then costs can be estimated for each project.

The source of the funding for the Master Watershed Study is a Legislative Pre-Disaster Mitigation (LPDM) FEMA grant.

1.3 Scope of Study

The scope of the study is to provide an existing and ultimate development condition open channel study of the above mentioned creeks. The following items summarize the general tasks performed during the study:

1. Data Collection
2. Site Visits / Field Observations
3. Hydrologic Parameter Development
4. Hydrologic Model Development
5. Storm Event Analysis
6. Hydraulic Analyses
7. Flood Control – Identification of Areas of Concern
8. Environmental Permitting
9. Flood Control Solutions
10. CIP Recommendations

These general tasks are summarized as chapters in the Master Watershed Study report. Descriptions of the methodologies employed in the study and summary results are discussed in the body of the report, which is labeled as Volume I. Detailed figures of the hydrologic parameters are presented in Appendix A in Volume II of the report. Detailed tables of calculations are presented in Appendix B. Digital files such as GIS shapefiles, hydrologic and hydraulic model output, and other data are included on a CD in Appendix C. A full version of the Environmental and Cultural Resource Report performed by CP&Y as part of the Master Watershed Study is included as Appendix D.

1.4 Study Limits

The Master Watershed Study will consist of a detailed hydrologic and hydraulic analysis of several creeks and critical waterways within the city limits. The creeks to be studied are:

1. San Felipe Creek (SFC), from its confluence with the Rio Grande floodplain upstream to the northern end of the San Felipe Springs Country Club;
2. Tributary of San Felipe Creek (SFCT1) in the Qualia, Alderete, and Las Vacas area;
3. Calaveras Creek (CVC), from its confluence with the San Felipe Creek mainstem eastward to Dividend near US 90;
4. Tributary to Calaveras Creek (CVCT1), also known as FEMA Stream 1, from its confluence with the Calaveras Creek mainstem upstream to Vitela Street;

5. Tributary to Cienegas Creek (CNCT1), also known as FEMA Stream 2, from its confluence with the Cienegas Creek mainstem upstream through the airport to near Main Street;
6. Cantu Branch (CB) of Cienegas Creek, from its confluence with the mainstem near Encino and Mary Lou upstream to the confluence of CBT6;
7. Tributaries of Cantu Branch (CBT1) (also known as FEMA Stream 3), CBT2, and CBT3 in the northwest part of Del Rio ending in the Kings Way / Amistad area;
8. Tributary of Cantu Branch (CBT4) in the north central part of Del Rio near the Dodson area;
9. Tributary of Cantu Branch (CBT5) in the north part of Del Rio crossing 277/377 north of Agarita.

1.5 Existing FEMA Study

FEMA produced an update to the Val Verde County Flood Insurance Study (FIS) in July 2010. The FIRM updates in July 2010 only affected the tie-ins to the surrounding Val Verde County FIRMs. There were no updates to the hydrologic or hydraulic models used for the City of Del Rio by the FEMA contractor. The FIS update was specific to the maps surrounding the City of Del Rio. The San Felipe Creek watershed consists of San Felipe Creek, Calaveras Creek, and Stream 1 that are partially mapped as studied Zone AE regulatory floodplains. The existing floodplain is depicted on the Flood Insurance Rate Map (FIRM) panel 48465C1735D, effective July 22, 2010. Calaveras Creek is also on FIRM panel 48465C1755D. The Cienegas Creek watershed consists of Cantu Branch, Stream 2, and Stream 3 that are partially mapped as studied Zone AE regulatory floodplains. The existing floodplain is depicted on the Flood Insurance Rate Map (FIRM) panel 48465C1645D, effective July 22, 2010.

The FIRM panels are presented at the end of this chapter.

2.0 SITE OBSERVATIONS AND DATA COLLECTION

2.1 Data Sources

There were several sources of data used in the Master Watershed Study, both public domain and commissioned.

The public domain sources include all of the inputs used for the hydrologic data. These are described more fully in Chapter 3. The topographic data used to delineate watersheds came from the USGS and IBWC LiDAR. The soil cover information was obtained from NRCS. Impervious cover was developed from land use provided by the City of Del Rio. Up to date aerial photography was also provided by the City of Del Rio. Design precipitation depths were taken from a USGS publication for Texas.

Background information on previous modeling efforts was provided by the Fort Worth District of the USACE. Effective floodplain maps are available from FEMA. There were no legible copies of legacy hydraulic models available from the FEMA library. Historical flows for Cienegas Creek and San Felipe Creek have been published by IBWC. Historic precipitation records were obtained from the National Weather Service.

Data commissioned for this study include the physical survey of existing roadways, culverts, and bridge structures. Representative channel sections were also surveyed.

2.2 Source of LIDAR

2009 LIDAR data was supplied by International Boundary and Water Commission (IBWC) and USGS DEM data. These data were used to delineate the watershed and subbasins, as well as, supplement the hydraulic cross sections between the surveyed sections. In general, the LIDAR data compares very well to the acquired survey data.

The horizontal coordinates and elevations are based on Texas State Plane Coordinate System, Texas south Zone 4204, North American Datum 1983 (NAD83). The vertical datum for the study is NGVD 1988.

2.3 Field Survey Data

A master plan level field survey was conducted by Bain Medina Bain along the waterways in this study. The primary objective of the survey acquisition is to provide additional information and detail for various roadway and hydraulic structures with no available post construction plans. The survey data is also used to supplement the LIDAR data at various natural cross sections within the creeks. The natural cross section data is necessary to better define then channel invert because the LIDAR data does not typically capture information below the water level. Additionally, the survey data at the natural cross section locations are necessary to provide necessary detailed information in the flood prone overbank areas. Finally, the surveyor included pictures at each of the crossings to document the overall condition of the structure.

2.4 Site Investigations

CP&Y staff visited Del Rio many times to collect information about critical flooding locations. Staff also walked the majority of the studied creeks. Public access was available for almost all of the creeks within the City Limits. Where the creeks crossed large privately owned tracts, access to the creeks were limited to public roads.

CP&Y collected field notes, sketches, photographs and maps on at least 10 different days from June 2010 through July 2012. The comprehensive stream reconnaissance occurred during January 2011. Environmental mapping occurred in July 2012 after project sites were proposed.

The site visits provided reference photographs that were used to develop channel roughness coefficients for the hydraulic modeling. Sketches and measurements of bridges and culverts provided comparisons against the survey.

Over one thousand photographs were taken during the site visits. While not reproduced here, copies of the digital pictures are included on the Data CD in **Appendix C** in **Volume II**.

3.0 HYDROLOGIC PARAMETER DEVELOPMENT

3.1 Introduction

The hydrologic analysis for the San Felipe and Cienegas Creek watersheds was developed using methods outlined in the US Department of Agriculture Soil Conservation Service (SCS) publication Technical Release No. 55 (TR-55) *Urban Hydrology for Small Watersheds*, and guidelines from the San Antonio River Authority (SARA). A number of hydrologic parameters were calculated based on the physical characteristics of the watershed to develop the watershed hydrology. In particular, sub-basin areas, rainfall depths, times of concentration, initial abstractions and loss rates, impervious coverage, and routing parameters were calculated to determine runoff hydrographs and the timing effect of routing through the watersheds.

The hydrologic analysis was necessary to develop peak discharges at various crossings and study points within each watershed based on physical characteristics of the contributing sub-basin areas. The analysis was developed for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence interval storms. The subsequent peak discharges were developed for existing and ultimate watershed conditions, and were then used in the hydraulic analysis to assess the impact of the flooding situation along the creeks within the City.

3.2 Drainage Area Delineations

The contributing watersheds to San Felipe Creek and Cienegas Creek are characterized by prominent high ridges and canyons located in the northern and upper parts of the respective watersheds. These canyons ultimately expand and flatten into wider floodplain valleys as they approach the Rio Grande River floodplain through the City of Del Rio. Many of the tributaries contributing to the major creeks have poorly defined channels or merely a concentration of shallow flow. In addition, the contributing watersheds experience a significant amount of elevation drop ranging from 350 to 400 feet from the upper extent of the watersheds into the City.

The Cienegas Creek and San Felipe Creek watersheds were sub-divided into forty-five (45) and fifty (50) sub-basins, respectively. These drainage sub-basins are necessary to quantify contributing drainage area and peak discharges at various crossing locations and study points. Drainage areas were delineated using LIDAR data obtained from the IBWC and supplemented with 10-ft USGS topographic data in the upper portion of the watersheds. Field investigation was used to further refine the sub-basin boundaries. The contributing drainage areas at the terminus of the study (confluence with the Rio

Grande River) for the Cienegas Creek and San Felipe Creek watersheds are approximately 23.6 and 53.0 square miles, respectively. **Figure A-1** in **Appendix A** shows a detailed version of the entire Drainage Area Map for the Del Rio Master Drainage Study. **Figure 3-1** at the end of this chapter shows smaller scale versions of the drainage areas and watersheds.

3.3 Time of Concentration

The time of concentration (Tc) for each sub-basin was calculated using the methods and procedures outlined in TR-55. The existing condition Tc within each sub-basin was determined by the summation of three (3) different flow components: sheet flow, shallow overland flow, and channel flow. It should be noted that no adjustment was made to the Tc for the ultimate condition because the overall change in runoff volume was determined to be insufficient to impact flow velocities in the channel component.

The majority of sheet flow lengths were limited to 100 ft. However, a few lengths were determined to be slightly longer based on available aerials and topographic data. The Manning's "n" retardance value for overland sheet flow are generally representative of mowed lawns in the developed areas of the City and range grass or shrubs in the rural parts of the upper watersheds north of the City. The 2-year 24-hour rainfall depth was taken from USGS rainfall data for Val Verde County.

The shallow overland flow component is characterized by concentrated flow that travels across land (unpaved or paved) in an area where no clearly defined conveyance collection or channel exists. Many of the sub-basins around Del Rio are unique in that unusually long components of shallow overland flow are present. Shallow concentrated flow lengths usually range from about 1,500 to 2,500 feet. However, several of the sub-basins within the watersheds have flow lengths that exceed 9,000 to 10,000 feet. The longer duration Tc values are indicative of longer shallow overland flow paths and slower velocities.

The channel flow component is generally determined where a natural or open channel (ditch) is clearly present from field observation or visible on aerial and topographic data. In most sub-basins, channel flow begins at the terminus of the shallow overland flow. The channel flow velocities were determined with the Manning's equation in conjunction with a representative geometric cross section taken from the available topographic data. The Rational Method was used to approximate a 2-year peak discharge for the contributing sub-basin. The flow velocities were solved using a normal depth calculation with the Haested Method computer program Flow Master. The flow paths for the Tc calculations for the subwatersheds in the San Felipe Watershed are shown in **Figure A-2** in **Appendix A**. Similarly, flow paths for the Tc calculations for the subwatersheds in

the Cienegas Watershed are shown in **Figure A-3** in **Appendix A.** The summary of all of the hydrologic parameters used in the hydrologic models for the San Felipe Creek watershed are presented in **Table 3-1** at the end of this chapter. The parameters tabulated in the table are Drainage Area ID, drainage area, NRCS soil curve number (CN), the existing percent impervious cover, the ultimate percent impervious cover, the existing conditions lag time and the ultimate conditions lag time. The other parameters will be discussed in the following sections in this chapter. The summary of the hydrologic parameters for Cienegas Creek watershed are presented in **Table 3-2**. Detailed versions of the time of concentration and lag time calculations by subwatershed are presented in **Tables B 3-1** through **B 3-4** in **Appendix B**.

3.4 Infiltration Loss and Initial Abstraction

Infiltration loss rates and initial abstractions were calculated using the NRCS Curve Number (CN) method outlined in TR-55. Weighted open space CN values for each sub-basin were computed with a geographic information system (GIS) using hydrologic soil groups along with respective open space CN values. Because the average annual rainfall for the City of Del Rio is only 18.5 inches, the average moisture condition (AMC) was determined to be an AMC-II. This set the boundary conditions for the CN values in the hydrologic models.

The average vegetative conditions are predominately fair or poor due to the lack of rainfall. However, some of the areas within the riparian zone and spring fed (or irrigated) areas along San Felipe Creek were determined to be in good condition. The open space CN values were assigned according to vegetative and antecedent moisture condition.

The study area consists predominantly of hydrologic soil groups B and D with localized areas of group A soils within the riparian zone. The variation of soils within the watershed, as well as other physical and compositional characteristics of the soils, was determined using digital files obtained from USDA-NRCS Soil Survey Division's Soil Map for Val Verde County, Texas. **Figure A-4** in **Appendix A** shows the soil group classifications for the San Felipe Creek watershed. **Figure A-5** in **Appendix A** shows the soil group classifications for the Cienegas Creek watershed. The hydrologic soil classifications for each subarea in the San Felipe Creek watershed are shown in **Figure A-6** in **Appendix A**. Similarly, the hydrologic soil classifications for each subarea in the Cienegas Creek watershed are shown in **Figure A-7** in **Appendix A**. As mentioned before, the final weighted curve numbers (CNs) for each watershed are summarized in **Tables 3-1** and **3-2** at the end of this chapter. A tabulation of the weighted open space CN calculations for each sub-basin is presented in **Table B 3-5** in **Appendix B**.

3.5 Percentage of Impervious Surface

Land use data were obtained from City zoning and parcel data to determine the percentage of impervious surface for each sub-basin. The watersheds are comprised predominantly of four types of land use: residential (both 1/4 acre and 1/8 acre), commercial, facilities, and limited amounts of open space. **Table B 3-6** in **Appendix B** shows the land use codes, descriptions, and the respective percentage of impervious surface for existing conditions in the San Felipe Creek watershed. **Table B 3-7** in **Appendix B** shows the same information for existing conditions in the Cienegas Creek watershed. **Tables B 3-8 and B 3-9** in **Appendix B** present the impervious cover information for ultimate conditions in the San Felipe and Cienegas Creek watersheds respectively. **Figure A-8** in **Appendix A** depicts the existing land use data within the San Felipe Creek subwatersheds. Similarly, **Figure A-9** in **Appendix A** depicts the existing land use data within the Cienegas Creek subwatersheds. **Figure A-10** in **Appendix A** depicts the existing impervious cover data within the San Felipe Creek subwatersheds while **Figure A-11** in **Appendix A** illustrates the existing impervious cover data within the Cienegas Creek subwatersheds. Tabulations of the weighted percent impervious values for each sub-basin are located in **Tables 3-1** and **3-2**.

3.6 Rainfall Distribution and Hyetographs – May 24, 2010 Storm

Precipitation data used in the hydrologic models were based on a 24-hour duration storm using the hypothetical NRCS Type-II rainfall distribution. The precipitation data was obtained from the USGS Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas. The 24-hour precipitation depths used to develop the rainfall hyetograph for various recurrence interval storms (2-year through 500-year) are shown in **Table 3-3**.

Table 3-3: Precipitation Depths

Recurrence Interval Storm	Depth (in)
2-year	2.9
5-year	4.25
10-year	5.2
25-year	6.4
50-year	7.4
100-year	8.6
500-year	11.0

In addition to the to the standard rainfall hyetograph simulations, CP&Y developed a hyetograph simulation using rainfall data collected from the National Weather Service for the May 24, 2010 storm. This particular storm produced approximately 7.2 inches of rainfall in 2 hours. It should be noted that other rainfall gages (Coco RAS) around the City and within the upper part of the watershed recorded rainfall depths in excess of 9 to 10 inches. For the purposes of this rainfall event analysis, no aerial reduction was assumed. Based on depth and duration, this storm equates to a 250-year recurrence interval storm. **Table 3-4** shows the 15 minute interval cumulative precipitation depths from the May 24, 2010 storm.

Table 3-4: May 24, 2010 Storm Data (NWS)

Time (min)	Cumulative Depth (in)
0	0
15	0.83
30	2.24
45	3.48
60	3.48
75	4.48
90	5.27
105	6.46
120	7.17

3.7 Areal Reduction – SARA Method

For larger storm events that cover a greater physical area, it is reasonable to assume that there is a non-uniform spatial distribution of precipitation over the entire watershed. That is, larger watersheds that are in excess of 10 square miles are more likely experience some overall reduction in total precipitation depth based on the area of the watershed relative to the most intense area of the storm event. The areal reduction values and interpolation procedures used in this study are outlined in the (SARA) guidelines.

Hydrographs with varying rainfall depths were developed in 10 square mile increments up until the total storm area exceeded the area of the watershed. Peak flow values for all recurrence interval storms were interpolated for each hydrologic sub-basin and hydrograph combination point based on the contributing drainage area. A tabulation summary of the areal reduction for San Felipe Creek is located in **Table B 3-10** in

Appendix B. The areal reduction calculations for Cienegas Creek are found in **Table B 3-11** in **Appendix B**.

All single sub-basins are less than 10 square miles with no additional reduction in peak discharge from areal reduction. However, many of the hydrograph combination points (junctions) within the San Felipe Creek watershed experience a relatively significant reduction in peak flow because of contributing drainage areas are greater than 10 square miles and the benefit from areal reduction of a larger storm area. It should be noted that the studied tributaries and Cantu Branch within the Cienegas Creek watershed did not benefit from areal reduction because none of the combination points had contributing drainage areas greater than 10 square miles.

Areal reduction values are for reference purposes and were not used in the hydraulic model analyses.

3.8 Hydrograph Transformation

The NRCS Unit Hydrograph Method was used to produce a synthetic runoff hydrograph for each of the recurrence interval storms discussed above. The hydrograph is transformed using the rainfall hyetograph and the physical parameters developed for each sub-basin described in the previous sections.

3.9 Routing – Muskingum and Modified Puls

3.9.1 Muskingum-Cunge

The subsequent hydrographs were routed between sub-basins using the Muskingum-Cunge routing method (where modeled hydraulic cross section data were unavailable). The routing geometry was modeled as an approximate channel with eight (8) points that were developed from the available topographic data. The Manning's n-value for channel routing was estimated from aerial photography and field investigations. Interim flows were calculated using the Rational Method for each subarea. The proposed channel velocity was estimated using normal depth calculations software based on the Rational Method flow, basin slope, and estimated channel roughness. Detailed calculations for the San Felipe Creek subwatersheds are found in **Table B 3-12** in **Appendix B**. The same calculations for Cienegas Creek are presented in **Table B 3-13**.

3.9.2 Modified-Puls

The Modified-Puls method was used to route the sub-basin combined discharges through the studied channel segments using the newly developed hydraulic cross section data from this study. This method was employed because of the ease of acquiring storage-discharge relations from the HEC-RAS model. These data were taken between the upstream and downstream cross sections of the sub-basin through which the combined flows were routed. The sub-reach parameters were calculated using the 10-year computed median velocity along with the reach length and computational interval within the routing sub-reach.

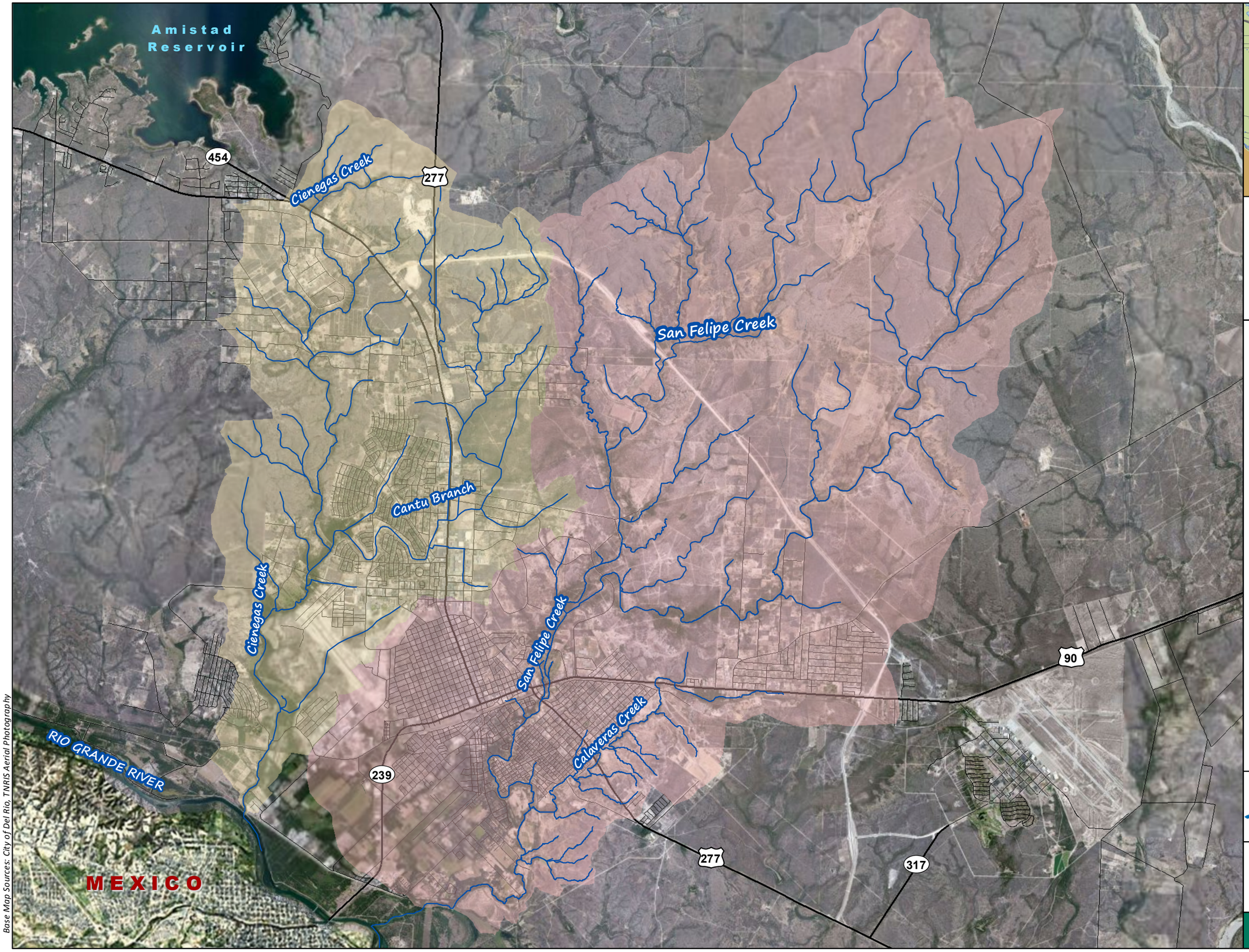
References:

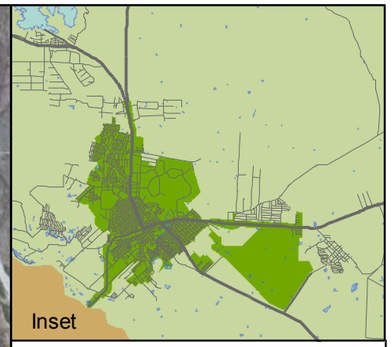
Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Report TR-55, U.S. Department of Agriculture, Washington DC, June 1986.

Asquith, W. and Roussel, M., Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, Scientific Investigations Report 2004-5041, U.S. Geologic Survey, Austin, TX, June 2004.

HEC-RAS Users Manual: U.S. Army Corps of Engineers, 2010.

Hydrologic Modeling System HEC-HMS Users Manual Version 3.5: U.S. Army Corps of Engineers, Hydrologic Engineering Center, August 2010.





Cienegas Watershed

San Felipe Watershed

Flowpath

0 0.5 1 Miles

Vicinity Map

FIGURE 3-1

Base Map Sources: City of Del Rio, TNRIS Aerial Photography

Table 3-1

SUMMARY OF HYDROLOGIC PARAMETERS

SAN FELIPE CREEK WATERSHED

Basin_ID	Area (SQMI)	NRCS CN	EXIST % IMP	ULT % IMP	Lag Time (min)
CVC-05	1.6665	74.3	30.9	50.8	83.8
CVC-10	0.7645	74.4	22.5	61.4	24.3
CVC-15	0.1889	73.0	21.7	73.4	19.8
CVC-20	0.8222	74.4	3.8	47.3	43.3
CVC-25	0.2821	70.5	15	60.8	15.4
CVC-30	0.3159	72.7	24.2	72	12.9
CVC-35	0.0932	67.7	24.2	59.9	10.9
CVC-40	0.1233	81.0	4.8	69.4	9.2
CVC-45	0.3659	72.0	22.6	30	26.4
CVC-50	0.4508	73.4	11.5	12.4	13.7
CVCT1-05	0.3862	64.3	80.7	80.7	46.4
CVCT1-10	0.0921	62.4	83.3	83.3	7.7
CVCT1-15	0.0473	62.0	84.1	84.1	7.9
SFC-05A	4.613	76.8	0	0	58.1
SFC-05B	3.7308	72.8	0	0	75.5
SFC-05C	6.1301	68.9	2	12.7	80.6
SFC-05D	1.2367	67.3	0	40.8	36.9
SFC-05E	1.1691	70.4	0	29.5	72.1
SFC-05F	0.4011	72.6	0.7	35.6	53.1
SFC-10A	6.83	76.1	0	0	68.7
SFC-10B	4.5874	72.2	0	3.3	41.6
SFC-10C	6.1118	65.8	1	29.7	52.6
SFC-10D	0.9227	63.1	0.7	25.6	52
SFC-15	1.1072	64.0	3.9	7.6	37.8
SFC-20	1.2469	67.5	24.2	32.5	28.4
SFC-25	1.1804	62.9	73.4	73.8	31.1
SFC-30	0.2539	61.9	78.1	78.1	23.3
SFC-35	0.2033	61.7	60.6	60.6	17.5
SFC-40	0.1575	61.7	64.6	65.4	20.3
SFC-45	0.1254	61.2	43.9	48.6	15.9
SFC-50	0.1872	62.1	36.9	43.6	14.8
SFC-55	0.1002	62.0	77.7	77.7	12.3
SFC-60	0.2758	65.5	3.3	4.4	29.8
SFC-65	0.812	71.1	12.9	12.9	34.9
SFC-70	0.1574	64.0	9.2	9.2	16.1
SFC-75	0.9789	60.1	11.5	11.5	49
SFCT1-05	0.3786	66.7	75	75	13.5
SFCT1-10	0.289	64.0	61	73.3	13.7
SFCT1-15	0.1762	62.0	56.9	58	19.8
SFCT1-20	0.1332	62.0	46.4	59.1	15.4
SFCT1-25	0.3882	63.0	82.5	82.5	27.5
SFCT1-30	0.2969	68.5	45	70.9	21.2
SFCT1-35	0.1182	63.4	12.8	63.1	19.5
SFCT1-40	0.2152	70.8	30.7	74.5	38.3
SFCT1-45	0.87	62.6	16.7	26.6	80.8
SFCT1-50	0.5276	61.8	70.3	70.9	25.3
SFCT1-55	0.3796	61.5	44.1	66.3	40.7
SFCT1-60	0.3454	61.4	33.3	34	28.5
SFCT1-65	0.5684	61.1	28.1	39.8	54.3
SFCT1-70	0.2236	60.0	14.2	14.2	31
TOTAL	53.0278				

Table 3-2

SUMMARY OF HYDROLOGIC PARAMETERS

CIENEGAS CREEK WATERSHED

Basin_ID	Area (SQMI)	NRCS CN	EXIST % IMP	ULT % IMP	Lag Time (min)
CB-05	3.2655	71.7	3.8	23.5	41.8
CB-10	0.1308	72.2	10.8	58.4	15.3
CB-15	0.2286	54.1	6.3	41.6	69
CB-20	0.5728	58.8	9.1	31.9	75.9
CB-25	0.3482	60.7	0.3	32	74.7
CB-30	0.2472	68.4	60.4	62.4	30.5
CB-35	0.425	66.4	1.9	22.1	63.8
CB-40	0.3247	73.4	5.3	8.7	37.4
CB-45	0.2276	72.4	60.4	62.8	18.2
CB-50	0.1852	63.6	49.1	58.7	18.3
CB-55	0.2247	68.8	63.9	63.9	22.2
CB-60	0.4519	71.7	54.6	54.6	25.5
CB-65	0.278	68.0	44.6	44.6	19.9
CB-70	0.2304	70.5	34.8	43	20.7
CB-75	0.2061	73.1	17.4	29	35.4
CBT1-05	0.1136	79.9	40.6	44.6	18
CBT1-10	0.1387	74.5	47.9	47.9	15.3
CBT1-15	0.3594	75.2	52.3	54.3	20.3
CBT1-20	0.2123	71.2	36.9	36.9	25.2
CBT4-05	0.2375	75.0	54.7	55.4	24.8
CBT4-10	0.1347	66.9	74.3	74.3	16.8
CBT5-05	0.4751	74.9	22.3	43	34.7
CBT5-10	0.139	72.9	21.1	54	23.5
CNC-05	1.7881	76.2	0.2	7.9	40.6
CNC-10	2.6177	75.3	4.5	35.5	70.2
CNC-15	0.3604	78.9	1.8	45.4	16.6
CNC-20	1.1294	74.1	16.7	39.3	63.5
CNC-25	1.0097	78.1	2.5	22.8	47.1
CNC-30	0.4869	75.9	10.6	24.9	56.5
CNC-35	1.2746	79.5	1.1	30.7	30.7
CNC-40	0.2017	71.9	0.5	11.9	30.4
CNC-45	0.1567	72.3	17.4	26.5	23.2
CNC-50	0.3938	74.1	3.3	19.8	39.5
CNC-55	0.2606	64.4	4.1	15.7	29.4
CNC-60	0.1359	71.6	32.3	55.2	22.6
CNC-65	0.9041	71.9	3.2	28.7	43.4
CNC-70	0.6373	70.1	1.1	27.7	46.3
CNC-75	0.4948	72.3	5.3	25.3	37
CNC-80	0.3673	72.8	37.6	48.8	29.2
CNC-85	0.1007	74.1	39.4	58.5	21.5
CNC-90	0.8062	66.4	15.8	21.4	40.2
CNCT1-05	0.3259	67.3	60.9	60.9	22.4
CNCT1-10	0.1503	73.7	33.7	60.7	19.9
CNCT1-15	0.2666	70.8	19.1	44.2	27
CNCT1-20	0.5711	70.2	3.1	40.8	41.6
TOTAL	23.5968				

4.0 HYDROLOGIC METHODS AND RESULTS

4.1 Introduction and Software

The hydrologic parameters described in Chapter 3 were calculated for the subareas in the San Felipe Creek and Cienegas Creek watersheds. The watershed hydrology was analyzed using the HEC-HMS version 3.5, Hydrologic Modeling Software developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers in 2010. The hydrologic parameters were input into the model one subarea at a time. Drainage areas, routing times, NRCS curve numbers, the percent impervious cover, and the routing time from one subarea to another all are used by the HEC-HMS model to determine how much runoff occurs after a specified rainfall event.

4.2 HEC-HMS Model Setup

The HEC-HMS model was used to simulate flows for two sets of watershed conditions – existing and ultimate. Existing conditions corresponds to the year 2010. The land use information is from 2011, the aerial photography reflects 2010, the topography is based on LiDAR from 2009, and the channel survey is from 2012. Existing conditions represents the extent of city growth and development of the current city.

Ultimate conditions corresponds to the maximum future development. Very little open space is assumed. Discussions with City staff about projected development corridors and potential land uses have guided the assumptions about the Ultimate Conditions. In terms of the hydrologic model, only two parameters change from existing to ultimate conditions. The percent impervious cover increases and the lag time decreases. The percent impervious cover increases as the open space is converted into some form of development. The lag time decreases to reflect the higher runoff efficiency and lower surface roughness that normally slows runoff and promotes infiltration.

The main watersheds have been divided into many smaller subwatersheds. These subareas are numbered with low numbers from the upper portion of the watershed, and increase as the subareas move downstream. The numbering system initially has been set up with gaps of five, so that as the City develops and grows, larger subareas can be divided into up to five smaller units. The addition of letters can extend the system even further.

In the results tables that follow, the subareas are labeled with the Hydrologic Element from the HEC-HMS model. For example, the first subarea listed in the table is for CVC-05. The identifier CVC stands for Calaveras Creek. CVCT1 stands for Calaveras Creek Tributary 1. SFC is the identifier for San Felipe Creek, and SFCT1 stands for San Felipe

Creek Tributary 1. In the Cienegas Creek watershed, CNC stands for Cienegas Creek. CB stands for Cantu Branch, CBT1 stands for Cantu Branch Tributary 1, CBT4 stands for Cantu Branch Tributary 4, and CBT5 stands for Cantu Branch Tributary 5. Cantu Branch Tributaries 2 and 3 have lost their natural channels because of development in the City over the past 20 years.

The storms simulated in HEC-HMS for the Master Watershed Study are the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr storm events. For comparison purposes, the May 2010 storm event fell into the 250-yr return period. The 1998 flooding associated with Tropical Storm Charlie is off the chart – beyond a 500-yr event.

4.3 Hydrologic Model Results – Existing Conditions

Table 4-1 at the end of the chapter presents the simulated peak flows for existing watershed conditions for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr storm events for the subareas in the San Felipe Creek watershed. The results are presented for Calaveras Creek first, followed by Calaveras Creek Tributary 1, then San Felipe Creek mainstem, and finally San Felipe Creek Tributary 1. The results in Table 4-1 represent only the individual runoff amounts from each subarea in the watershed. The entire HEC-HMS output, for the existing conditions San Felipe Creek watershed, with hydrograph routing and combination points, is presented in **Table B 4-1** in **Appendix B**.

Similarly for the Cienegas Creek watershed, **Table 4-2** at the end of the chapter presents the simulated peak flows for existing watershed conditions for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr storm events. The results are presented for Cantu Branch first, followed by Cantu Branch Tributary 1, Cantu Branch Tributary 4, then Cantu Branch Tributary 5, Cienegas Creek mainstem and finally Cienegas Creek Tributary 1. The results in Table 4-2 represent only the individual runoff amounts from each subarea in the watershed. The entire HEC-HMS output for the existing conditions Cienegas Creek watershed, with hydrograph routing and combination points, is presented in **Table B 4-2** in **Appendix B**.

4.4 Hydrologic Model Results – Ultimate Conditions

Table 4-3 at the end of the chapter presents the simulated peak flows for Ultimate watershed conditions for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr storm events for the subareas in the San Felipe Creek watershed. The results are presented I the same order as for the existing conditions table. As before, the results in Table 4-3 represent only the individual runoff amounts from each subarea in the watershed. The entire HEC-HMS output, for the Ultimate conditions San Felipe Creek watershed, with hydrograph routing and combination points, is presented in **Table B 4-3** in **Appendix B**.

For the Cienegas Creek watershed, **Table 4-4** at the end of the chapter presents the simulated peak flows for Ultimate watershed conditions for the same storm events. As mentioned previously, the results in Table 4-4 represent only the individual runoff amounts from each subarea in the watershed. The entire HEC-HMS output for the existing conditions Cienegas Creek watershed, with hydrograph routing and combination points, is presented in **Table B 4-4** in **Appendix B**.

4.5 Comparison of Results

The results from the HEC-HMS models were compared to other sources of data to verify that the model results were within acceptable ranges of values. At various design points, calculations using the 2011 TxDOT OmegaEM Regional Regression Equations were performed. These TxDOT regression equations are not valid for urbanized watersheds, so the comparisons were for areas in the upper reaches of San Felipe Creek and Cienegas creek watersheds. The results were scattered, with some regression values above and some below the simulated results. Two limitations to the TxDOT OmegaEM regression equations is that are applicable to watershed greater than 10 sq. miles in size and very little impervious cover. Most of the Del Rio subareas are well below the limit, and for areas that are greater than 10 sq. miles in size, it begins to include urbanized areas.

A much better comparison is the simulated existing conditions flows and published FEMA FIS flows. These compared well. **Table 4-5** lists the FIS flows from the 2010 Val Verde County FIS at various locations. The existing conditions flow values simulated in HEC-HMS compare very well at locations where the drainage areas are similar in size. For example, at the bottom of Calaveras Creek the 100-yr peak discharge is 7,300 cfs and the Master Watershed Study flow is 7,388 cfs. The variance is within 15% of the published FIS discharges, with the Study flows sometimes higher and sometimes lower for the San Felipe Creek watershed. Cienegas Creek was ignored in the FIS because in 1990 it was largely undeveloped. The hydrology from the original FIS was not updated in the 2010 reprint. As a consequence, the storm of 1998 was not discussed.

The impacts of these flows on the existing roads and bridges are discussed in Chapter 7.

References:

Asquith, W. and Roussel, M., Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, Scientific Investigations Report 2004-5041, U.S. Geologic Survey, Austin, TX, June 2004.

Hydraulic Design Manual, Texas Department of Transportation, October 2011.

Hydrologic Modeling System HEC-HMS Users Manual Version 3.5: U.S. Army Corps of Engineers, Hydrologic Engineering Center, August 2010.

TABLE 4-1
SUMMARY OF EXISTING CONDITIONS FLOWS - SAN FELIPE CREEK

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CVC-05	1.7	634	1031	1313	1671	1969	2326	3038
CVC-10	0.8	682	1120	1432	1826	2153	2544	3323
CVC-15	0.2	185	307	394	504	596	706	924
CVC-20	0.8	450	770	999	1288	1528	1815	2385
CVC-25	0.3	289	497	646	836	995	1184	1561
CVC-30	0.3	395	651	833	1064	1256	1485	1941
CVC-35	0.1	113	192	250	323	384	457	604
CVC-40	0.1	203	320	402	506	591	693	896
CVC-45	0.4	292	489	630	809	958	1136	1491
CVC-50	0.5	523	882	1138	1461	1729	2050	2688
CVCT1-05	0.4	266	404	501	625	729	853	1102
CVCT1-10	0.1	173	261	323	403	469	549	708
CVCT1-15	0.0	88	133	165	205	239	279	360
SFC-05A	4.6	2143	3609	4649	5963	7053	8356	10946
SFC-05B	3.7	1257	2213	2903	3780	4510	5386	7130
SFC-05C	6.1	1673	3106	4163	5521	6661	8034	10777
SFC-05D	1.2	387	838	1188	1650	2045	2526	3496
SFC-05E	1.2	374	678	900	1183	1420	1704	2271
SFC-05F	0.4	168	300	396	518	619	741	984
SFC-10A	6.8	2748	4661	6025	7746	9176	10885	14281
SFC-10B	4.6	2324	4131	5438	7099	8483	10140	13436
SFC-10C	6.1	2075	3986	5412	7254	8805	10673	14411
SFC-10D	0.9	280	560	773	1050	1284	1567	2137
SFC-15	1.1	457	887	1210	1630	1984	2411	3268
SFC-20	1.2	810	1419	1866	2440	2923	3503	4663
SFC-25	1.2	1012	1560	1953	2453	2872	3376	4384
SFC-30	0.3	267	409	510	639	747	878	1138
SFC-35	0.2	212	340	434	555	657	781	1029
SFC-40	0.2	158	250	318	404	477	565	742
SFC-45	0.1	97	167	223	296	359	437	597
SFC-50	0.2	192	325	423	548	653	780	1033
SFC-55	0.1	152	232	289	361	422	495	640
SFC-60	0.3	110	229	320	440	543	667	917
SFC-65	0.8	341	661	903	1221	1491	1818	2477
SFC-70	0.2	60	141	208	298	377	475	678
SFC-75	1.0	158	373	558	816	1045	1330	1924
SFCT1-05	0.4	556	848	1057	1321	1541	1805	2333
SFCT1-10	0.3	380	599	757	958	1126	1328	1731
SFCT1-15	0.2	180	288	366	466	550	651	852
SFCT1-20	0.1	143	235	303	390	464	552	728
SFCT1-25	0.4	386	584	724	902	1051	1230	1588

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
SFCT1-30	0.3	285	463	592	755	892	1057	1385
SFCT1-35	0.1	34	82	124	181	232	296	429
SFCT1-40	0.2	101	181	241	319	385	466	628
SFCT1-45	0.9	116	248	361	518	658	832	1196
SFCT1-50	0.5	495	771	970	1225	1438	1696	2213
SFCT1-55	0.4	181	309	407	535	644	777	1048
SFCT1-60	0.3	168	307	416	561	686	839	1153
SFCT1-65	0.6	136	257	359	498	621	774	1092
SFCT1-70	0.2	43	102	155	231	299	385	566

TABLE 4-2
SUMMARY OF EXISTING CONDITIONS FLOWS - CIENEGAS CREEK

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CB-05	3.294	1667	2935	3849	5011	5979	7144	9462
CB-10	0.1308	132	226	293	378	449	534	703
CB-15	0.2286	41	89	128	179	224	279	391
CB-20	0.5728	123	247	342	467	574	705	968
CB-25	0.3482	71	149	208	286	353	433	597
CB-30	0.2472	203	318	401	505	592	697	907
CB-35	0.425	129	245	330	440	533	645	868
CB-40	0.3247	189	325	423	547	651	774	1020
CB-45	0.2276	265	410	514	644	753	884	1145
CB-50	0.1852	182	296	379	485	573	680	893
CB-55	0.2247	229	357	448	564	660	776	1007
CB-60	0.4519	419	659	829	1045	1225	1441	1872
CB-65	0.278	272	441	562	717	846	1001	1311
CB-70	0.2304	213	351	450	576	682	808	1059
CB-75	0.2061	103	187	249	329	396	478	640
CBT1-05	0.1136	137	212	264	330	385	451	582
CBT1-10	0.1387	174	273	343	432	506	594	770
CBT1-15	0.3594	397	618	774	972	1136	1332	1724
CBT1-20	0.2123	166	277	357	460	546	649	855
CBT4-05	0.2375	234	363	455	570	667	782	1011
CBT4-10	0.1347	169	259	323	404	471	553	715
CBT5-05	0.4751	329	539	688	877	1034	1221	1594
CBT5-10	0.1105	94	156	201	257	304	360	472
CNC-05	1.7881	1045	1767	2280	2927	3464	4105	5379
CNC-10	2.6177	1022	1734	2241	2882	3415	4053	5321
CNC-15	0.3604	404	659	838	1063	1250	1472	1914
CNC-20	1.1294	281	552	761	1037	1273	1563	2150
CNC-25	1.0097	569	938	1198	1526	1798	2123	2768
CNC-30	0.4869	233	388	498	637	753	891	1166
CNC-35	1.2746	997	1621	2058	2607	3062	3606	4686
CNC-40	0.2017	94	184	251	337	411	499	675
CNC-45	0.1567	105	189	251	331	398	479	640
CNC-50	0.3938	211	370	484	629	750	895	1182
CNC-55	0.2606	71	168	246	350	441	552	782
CNC-60	0.1359	111	187	242	312	370	440	580
CNC-65	0.9041	417	752	996	1307	1567	1878	2499
CNC-70	0.6373	218	430	588	794	968	1178	1598
CNC-75	0.4948	207	398	541	727	885	1075	1456
CNC-80	0.3673	256	429	555	716	851	1014	1340
CNC-85	0.1007	90	148	190	244	289	342	450
CNC-90	0.8062	250	507	707	976	1207	1491	2068

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CNCT1-04	0.0713	90	139	175	219	257	301	390
CNCT1-05	0.2546	273	429	540	681	799	940	1223
	DA							

TABLE 4-3
SUMMARY OF ULTIMATE CONDITIONS FLOWS - SAN FELIPE CREEK

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CVC-05	1.7	865	1357	1707	2149	2517	2958	3837
CVC-10	0.8	829	1277	1594	1995	2329	2728	3525
CVC-15	0.2	251	382	474	590	687	803	1035
CVC-20	0.8	578	912	1148	1447	1695	1993	2586
CVC-25	0.3	376	584	733	921	1078	1265	1640
CVC-30	0.3	491	748	930	1159	1350	1579	2036
CVC-35	0.1	143	225	284	358	420	494	642
CVC-40	0.1	231	345	426	528	612	714	916
CVC-45	0.4	304	500	641	819	967	1145	1499
CVC-50	0.5	525	884	1140	1463	1731	2052	2690
CVCT1-05	0.4	266	404	501	625	729	853	1102
CVCT1-10	0.1	173	261	323	403	469	549	708
CVCT1-15	0.0	88	133	165	205	239	279	360
SFC-05A	4.6	2143	3609	4649	5963	7053	8356	10946
SFC-05B	3.7	1257	2213	2903	3780	4510	5386	7130
SFC-05C	6.1	1834	3273	4328	5680	6816	8183	10913
SFC-05D	1.2	645	1099	1437	1874	2245	2693	3594
SFC-05E	1.2	495	825	1063	1366	1619	1923	2529
SFC-05F	0.4	226	369	472	602	711	841	1100
SFC-10A	6.8	2748	4661	6025	7746	9176	10885	14281
SFC-10B	4.6	2372	4180	5485	7143	8525	10179	13471
SFC-10C	6.1	2795	4793	6255	8131	9706	11603	15400
SFC-10D	0.9	401	710	939	1235	1485	1786	2390
SFC-15	1.1	477	909	1232	1651	2004	2431	3286
SFC-20	1.2	869	1480	1927	2499	2980	3557	4712
SFC-25	1.2	1015	1563	1956	2456	2875	3379	4387
SFC-30	0.3	267	409	510	639	747	878	1138
SFC-35	0.2	212	340	434	555	657	781	1029
SFC-40	0.2	159	252	319	406	479	567	744
SFC-45	0.1	105	177	233	307	370	448	608
SFC-50	0.2	203	337	434	559	664	790	1043
SFC-55	0.1	152	232	289	361	422	495	640
SFC-60	0.3	112	231	322	442	545	669	919
SFC-65	0.8	341	661	903	1221	1491	1818	2477
SFC-70	0.2	60	141	208	298	377	475	678
SFC-75	1.0	158	373	558	816	1045	1330	1924
SFCT1-05	0.4	556	848	1057	1321	1541	1805	2333
SFCT1-10	0.3	411	631	788	988	1155	1356	1757
SFCT1-15	0.2	181	289	367	467	551	652	854
SFCT1-20	0.1	158	252	320	406	479	567	742
SFCT1-25	0.4	386	584	724	902	1051	1230	1588

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
SFCT1-30	0.3	362	557	696	872	1019	1196	1548
SFCT1-35	0.1	109	174	222	286	341	407	543
SFCT1-40	0.2	177	273	342	431	505	594	774
SFCT1-45	0.9	155	296	413	574	715	891	1256
SFCT1-50	0.5	498	773	973	1227	1441	1699	2215
SFCT1-55	0.4	249	393	499	636	752	892	1175
SFCT1-60	0.3	170	310	419	564	689	842	1155
SFCT1-65	0.6	177	308	414	558	683	837	1156
SFCT1-70	0.2	43	102	155	231	299	385	566

TABLE 4-4
SUMMARY OF ULTIMATE CONDITIONS FLOWS -CIENEGAS CREEK

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CB-05	3.294	2130	3423	4348	5520	6496	7667	10002
CB-10	0.1308	173	269	337	424	496	582	755
CB-15	0.2286	89	152	201	263	317	382	514
CB-20	0.5728	197	348	461	609	734	885	1190
CB-25	0.3482	129	226	298	391	470	566	757
CB-30	0.2472	206	321	403	507	594	699	908
CB-35	0.425	186	325	426	556	666	797	1060
CB-40	0.3247	192	329	426	550	654	777	1022
CB-45	0.2276	268	413	516	646	755	886	1147
CB-50	0.1852	195	310	392	498	586	692	904
CB-55	0.2247	219	346	438	554	650	767	999
CB-60	0.4519	419	659	829	1045	1225	1441	1872
CB-65	0.278	272	441	562	717	846	1001	1311
CB-70	0.2304	225	363	462	588	692	818	1068
CB-75	0.2061	115	200	262	341	408	489	650
CBT1-05	0.1136	137	212	265	331	386	452	583
CBT1-10	0.1387	178	277	347	435	509	597	772
CBT1-15	0.3594	390	612	769	967	1131	1328	1720
CBT1-20	0.2123	166	277	357	460	546	649	855
CBT4-05	0.2375	242	371	462	577	673	787	1016
CBT4-10	0.1347	169	259	323	404	471	553	715
CBT5-05	0.4751	350	572	731	930	1096	1295	1690
CBT5-10	0.1105	113	177	222	280	328	386	500
CNC-05	1.7881	1246	1955	2457	3090	3616	4246	5502
CNC-10	2.6177	1134	1909	2461	3158	3738	4432	5810
CNC-15	0.3604	467	727	909	1139	1329	1557	2011
CNC-20	1.1294	386	678	899	1187	1434	1733	2339
CNC-25	1.0097	642	1028	1300	1642	1927	2267	2943
CNC-30	0.4869	247	401	511	649	764	901	1175
CNC-35	1.2746	1162	1823	2288	2872	3357	3936	5090
CNC-40	0.2017	106	197	264	350	423	510	685
CNC-45	0.1567	114	198	260	340	407	487	647
CNC-50	0.3938	232	391	504	648	768	911	1196
CNC-55	0.2606	121	225	305	412	503	615	844
CNC-60	0.1359	100	176	231	302	362	434	577
CNC-65	0.9041	599	946	1193	1508	1771	2086	2714
CNC-70	0.6373	312	543	713	933	1117	1340	1788
CNC-75	0.4948	265	467	617	809	971	1166	1556
CNC-80	0.3673	278	452	578	738	873	1035	1359
CNC-85	0.1007	79	137	180	234	279	333	441
CNC-90	0.8062	465	752	963	1238	1470	1752	2320

HYDROLOGIC ELEMENT	DA (sq mi)	2-yr (CFS)	5-yr (CFS)	10-yr (CFS)	25-yr (CFS)	50-yr (CFS)	100-yr (CFS)	500-yr (CFS)
CNCT1-04	0.0713	90	139	175	219	257	301	390
CNCT1-05	0.2546	273	428	538	679	797	938	1220
CNCT1-10	0.1503	171	264	329	412	482	565	730
CNCT1-15	0.2666	242	389	494	627	738	870	1136
CNCT1-20	0.5711	346	569	730	935	1106	1312	1723

TABLE 4-5 COMPARISON OF STUDY FLOWS TO FIS DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA-*FIS* (sq. mi.)	DRAINAGE AREA-*Study* (sq. mi.)	PEAK DISCHARGES (cfs) 10 - Year Event (FIS)	10 - Year Event (CP&Y)	50 - Year Event (FIS)	50 - Year Event (CP&Y)	100 - Year Event (FIS)	100 - Year Event (CP&Y)	500 - Year Event (FIS)	500 - Year Event (CP&Y)
CALAVERAS CREEK										
At confluence with San Felipe Creek	5.56	5.60	4,350	3,362	6,400	5,593	7,300	7,388	9,600	8,854
At U.S. Route 277	3.98	4.13	3,650	2,780	5,300	4,561	6,100	5,176	8,000	6,988
At U.S. Route 90	2.64	2.62	2,900	1,891	4,650	2,601	4,650	2,855	6,000	4,844
CANTU CREEK										
At confluence with Cienegas Creek	9.47	8.13	6,500	2,666	9,900	6,120	11,300	8,110	15,400	10,280
At U.S. Route 277 & 90	6.37	6.57	5,200	2,543	7,800	6,377	9,100	8,300	12,200	9,940
At a point approximately 1.8 miles upstream of U.S. Routes 277 & 90	3.93	3.88	3,600	4,654	5,500	7,202	6,300	8,587	8,500	11,346
SAN FELIPE CREEK										
At confluence with Rio Grande	46	53.03	28,000	23,887	47,000	38,459	53,000	47,095	64,000	63,011
Upstream of Calaveras Creek	39	40.29	25,900	19,734	43,500	32,772	49,500	39,902	60,000	55,012
At a point approximately 0.6 miles upstream of U.S. Route 90	36.4	36.84	23,300	19,480	40,000	32,231	44,000	39,998	53,000	55,719
At a point approximately 0.6 miles downstream of the corporate limits of the City of Del Rio*	35.8		**		**		42,100		**	
An unnamed ranch road crossing*	35.6		**		**		41,900		**	
At a point approximately 1,760 feet downstream of upstream corporate limits of the City of Del Rio	18.4	18.45	**	13,721	**	21,987	23,700	27,551	**	37,947
At a point approximately 480 feet downstream of upstream corporate limits of the City of Del Rio	18.2		**		**		23,400		**	
At a point approximately 250 feet upstream of upstream corporate limits of the City of Del Rio	17.7	17.53	**	13,320	**	21,663	22,700	26,469	**	36,108
SAN FELIPE CREEK TRIBUTARY										
An unnamed ranch road crossing*	35.6		**		**		41,900		**	
At a point approximately 3,360 feet downstream of upstream corporate limits of the City of Del Rio	17	17.28	**	10,511	**	17,491	20,900	21,186	**	28,256
At a point approximately 1,760 feet downstream of upstream corporate limits of the City of Del Rio	16.9		**		**		20,700		**	
At a point approximately 800 feet downstream of upstream corporate limits of the City of Del Rio	15.6	15.71	**	10,003	**	16,539	19,500	19,973	**	26,559
STREAM 1										
At confluence with Calaveras Creek	0.58	0.53	950	570	1,300	841	1,450	995	1,850	1,297
At U.S. Route 277	0.43	0.39	800	501	1,100	729	1,250	853	1,550	1,102
STREAM 2										
At downstream corporate limits of the City of Del Rio	0.68	0.74	1,150	1,016	1,600	1,421	1,800	1,637	2,250	2,056
STREAM 3										
At confluence with Cantu Branch	0.8	0.82	1,250	1,558	1,750	2,332	1,950	2,761	2,500	3,616

* Located within area of crossover flow between San Felipe Creek and San Felipe Creek Tributary

** Data Not Available

5.0 HISTORICAL STORM EVENT ANALYSIS

5.1 Historical Precipitation Events

Daily precipitation records are available from the National Climatic Data Center (NCDC) for the period from December 1905 through the present. The records come from three recording sites: Station ID 412357 from 1905 to1951 and again from 1957 to 1963; Station ID 412360 from 1951 to 1957; and the present site at the Del Rio International Airport Station KDRT from 1963 onward. All of this data has been used as one data source even though the gage has moved a little throughout history.

The vast majority of the precipitation data is available only as 1-day rainfall total depths. The analyses described in this chapter reflect 38,987 recordings over the 107 year period. All "Trace" amounts were replaced with a value of 0.00 inches of rainfall.

5.2 Daily Precipitation Statistics and Frequency Analyses

Of the 38,987 daily recorded amounts, 6,474 were non-zero events where at least 0.01 inch of rainfall was recorded. This means that 17% of the days had some recordable amount of precipitation, and 83% of the time the weather was dry.

There were 1,146 days where the precipitation was 0.50 inches or more for the day. This means that approximately 3% of the time it was wet and 97% of the time there was little or no runoff.

For the other extreme, the largest 1-day precipitation depth was on August 23, 1998 when 17.03 inches of rainfall was recorded at the Airport gage. The next largest rainfall depth recorded was in June 1935 of 8.79 inches. **Table 5-1** below summarizes the ten largest rainfall events in Del Rio.

Table 5-1
Largest 1-Day Rainfall Events

Date	Amount (in.)
8/23/1998	17.03
6/13/1935	8.79
10/4/1969	7.60
10/13/2005	7.59
5/24/2010	7.12
5/27/2003	6.53
10/3/1944	6.50
8/29/2008	6.09
10/4/1930	5.43
10/24/1941	5.42

Of the ten largest rainfall events, four occurred in the last 10 years of record. Seven of the ten events occurred in the August through October period.

Further analyses of the rainfall records as a whole provides the following information in **Table 5-2**.

Table 5-2
1-Day Rainfall Statistics for Del Rio

Amount (in.)	Cumulative Count	Incremental Count	Percent of Total	Percent of Increment
6.0 +	8	8	0.02%	0.02%
5.0 +	14	6	0.04%	0.02%
4.0 +	21	7	0.05%	0.02%
3.0 +	38	17	0.10%	0.04%
2.0 +	142	104	0.36%	0.27%
1.0 +	519	377	1.33%	0.97%
0.5 +	1146	627	2.94%	1.61%
0.01 +	6474	5328	16.6%	13.7%
Zero +	38987	32513	100.0%	83.4%

These rainfall statistics do not match the design rainfalls depths discussed in Chapter 3. The values in the hydrology section used to develop the design flows for flooding events are based on annual exceedance rates, i.e., the maximum rainfall depths likely to occur in any given year. The statistics above are indicative of any given day, regardless of the time period. For the less frequent events, the actual statistics support the Annual Exceedance Probability (AEP). For example, the 100-yr (1% AEP) rainfall depth given in Chapter 3 from the USGS study is 8.6 inches for a 24-hr storm duration. According to the 107 year record, there has been only one event near that (8.79 inches). One event out of 107 years is slightly less than 1 percent. Similarly, the 50-yr (2% AEP) is given as 7.4 inches. There are two events out of 107 that are near 7.4 inches (7.60 and 7.59).

5.3 Historical Flooding in Del Rio

While there have been flooding in each decade in the history of Del Rio, the most deadly flooding occurred in August 1998 when the greatest rainfall depth occurred. The flooding event was caused by Tropical Storm Charlie, which sat over Val Verde, Kinney and Edwards Counties. The storm killed 10 people in Del Rio and another 6 people across the border in Ciudad Acuna. Over 17 inches of rainfall had fallen on Del Rio and surrounding Val Verde County on the 23rd, causing a wall of water to rush down San Felipe Creek. Some records show more than 24 inches fell on the two days of the 23rd and 24th together. More than 300 homes were damaged and eventually 1,600 people sought shelter in the Del Rio Civic Center.

More recently, in May 2010 an extreme rainfall event caused widespread flooding. Official rainfall records at the Del Rio Airport put the recurrence interval of the storm at greater than the 250-Yr event. This storm will be discussed in more detail in the following section.

5.4 May 2010 Storm Event

On May 24, 2010, an intense storm event occurred over most of the Del Rio watershed. The peak rainfall depths as recorded by the Airport showed 7.12 inches of rainfall in 120 minutes. This corresponds to a 250-yr storm event for the 2-hour storm duration based on data from USGS Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas. Fortunately there were no fatalities reported although there was intense flooding in the south part of the City. Highwater marks along fences showed 2-3 ft of sheet flow occurring along Alderete Street.

A simulation was performed in HEC-HMS using the recorded data from the airport. Unofficial storm depths recorded by CoCoRAS gages indicate fairly high precipitation depths east and north of Del Rio, so the assumption of uniform precipitation depth is more supportable.

5.5 2000's to 1950's Drought Comparison

The historical annual precipitation amounts are plotted on **Figure 5-1** at the end of this chapter. The average annual precipitation is 18.4 inches per year, based on the 106 complete years of record (1906-2011).

The annual precipitation plot shows that the drought of the 1950's is still the most critical dry period for the City of Del Rio. While 2001, 2006, and 2011 were all below normal years for rainfall, there were balanced out by the three wet years (2004, 2007, and 2010). **Table 5-3** presents the monthly and annual rainfall amounts for the historical Del Rio recording gages.

The data for this analysis is too voluminous to print in the report or the Appendix. The digital files will be included on the CD in Appendix C.

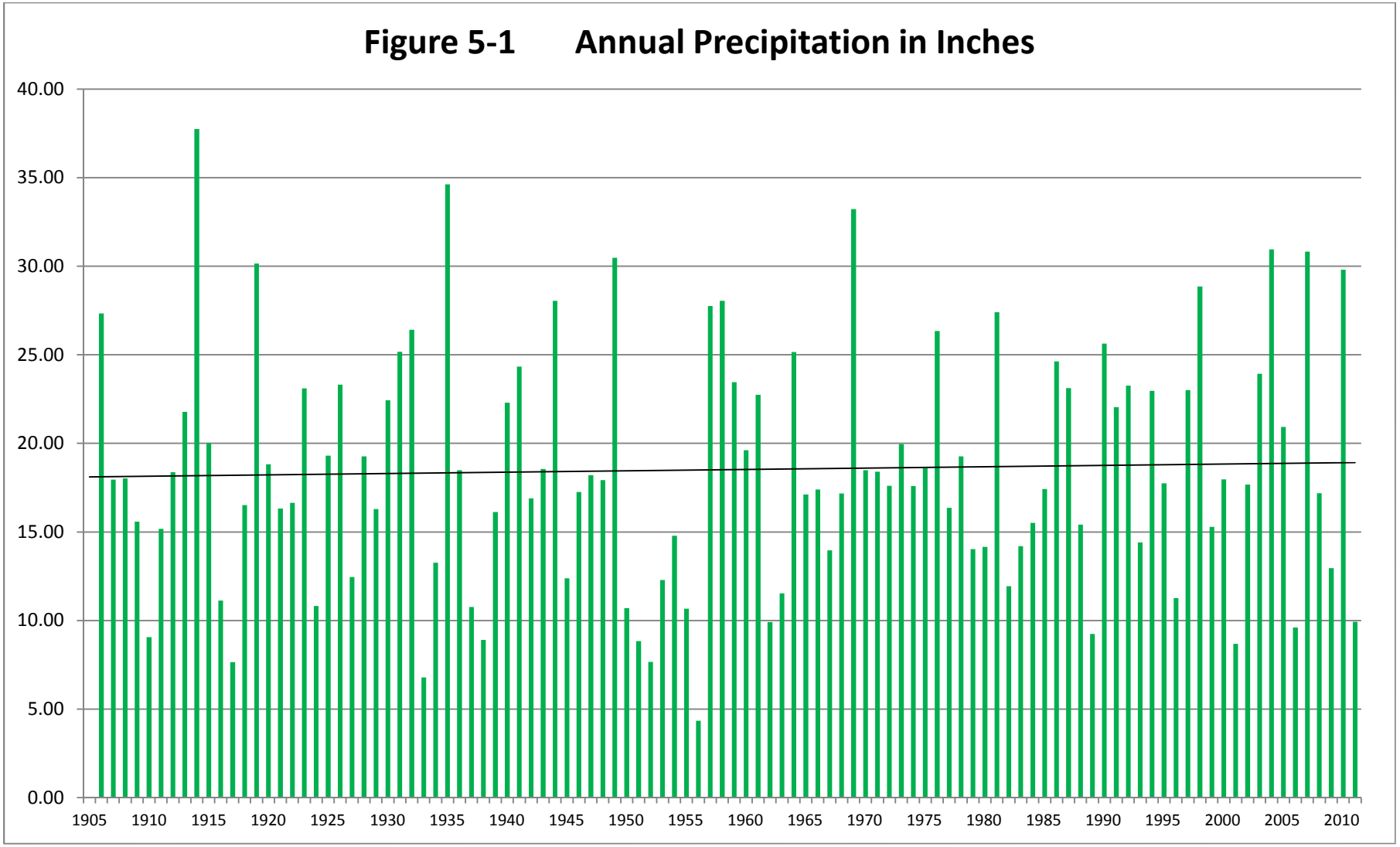


Figure 5-1 Annual Precipitation in Inches
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
1905
1910
1915
1920
1925
1930
1935
1940
1945
1950
1955
1960
1965
1970
1975
1980
1985
1990
1995
2000
2005
2010

TABLE 5-3
MONTHLY/ANNUAL/AVERAGE PRECIPITATION
DEL RIO TX (1905 - 2012)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	YEAR
1905														1905
1906	0.01	0.80	0.24	1.04	1.68	2.79	8.89	3.64	5.59	1.16	0.79	0.70	27.33	1906
1907	0.07	0.03	0.10	0.09	3.22	1.00	3.19	0.00	0.09	6.09	4.02	0.05	17.95	1907
1908	0.50	0.60	0.53	2.82	3.47	0.64	2.45	3.54	0.23	2.32	0.69	0.23	18.02	1908
1909	0.01	0.32	2.18	0.75	0.74	0.81	5.70	0.31	3.16	0.06	0.40	1.14	15.58	1909
1910	0.03	0.02	2.06	3.30	0.55	0.52	0.18	0.26	0.72	1.01	0.12	0.29	9.06	1910
1911	0.15	2.29	1.59	2.85	0.27	T	0.54	1.14	0.36	1.11	2.65	2.23	15.18	1911
1912	T	0.37	0.58	1.17	0.08	6.22	0.10	0.17	4.89	1.94	1.09	1.76	18.37	1912
1913	0.45	0.50	0.20	0.51	0.64	5.44	1.52	0.89	1.59	3.78	4.54	1.72	21.78	1913
1914	0.11	0.90	0.62	1.88	8.91	3.98	6.17	3.71	0.59	6.67	3.76	0.45	37.75	1914
1915	2.32	0.31	1.09	3.39	1.16	0.63	3.85	4.21	2.90	0.01	0.01	0.14	20.02	1915
1916	0.60	0.01	0.78	0.29	1.90	0.08	2.03	2.64	2.11	0.42	0.21	0.06	11.13	1916
1917	0.09	0.11	0.05	2.36	1.44	1.29	0.37	0.92	0.99	0.00	0.00	0.03	7.65	1917
1918	0.05	0.05	0.06	0.58	5.10	2.08	0.23	0.28	1.23	5.24	0.92	0.69	16.51	1918
1919	0.58	0.45	1.42	2.43	4.04	1.30	3.48	1.58	9.74	1.58	3.27	0.28	30.15	1919
1920	1.53	0.02	0.15	0.02	2.26	1.91	1.38	4.71	3.79	2.47	0.52	0.06	18.82	1920
1921	0.18	5.78	1.92	1.31	1.05	3.25	1.73	0.49	0.12	0.13	0.36	T	16.32	1921
1922	0.15	0.11	0.48	4.61	1.23	4.67	2.84	0.15	0.78	1.30	0.28	0.04	16.64	1922
1923	0.84	1.60	2.47	1.62	0.22	1.51	2.52	0.49	2.27	4.49	4.13	0.94	23.10	1923
1924	0.14	1.45	0.55	1.11	1.33	0.63	0.48	0.00	3.73	1.14	0.01	0.25	10.82	1924
1925	0.04	0.00	0.42	1.42	7.99	0.10	2.32	1.62	2.45	1.72	0.52	0.71	19.31	1925
1926	0.85	0.04	1.92	2.66	1.06	3.46	3.34	0.23	0.47	5.66	2.30	1.33	23.32	1926
1927	0.18	2.38	0.51	1.36	0.17	2.40	1.67	0.38	0.63	2.40	T	0.37	12.45	1927
1928	0.71	0.82	0.35	0.51	3.76	0.71	1.38	2.35	7.06	0.76	0.38	0.47	19.26	1928
1929	0.42	0.18	0.72	1.17	4.41	3.32	1.32	T	1.47	1.19	0.08	2.01	16.29	1929

TABLE 5-3
MONTHLY/ANNUAL/AVERAGE PRECIPITATION
DEL RIO TX (1905 - 2012)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	YEAR
1930	0.09	0.01	1.48	2.01	1.01	2.19	0.94	0.08	0.04	11.69	2.19	0.70	22.43	1930
1931	4.12	1.04	0.66	2.68	6.10	3.98	1.84	2.56	0.00	0.01	0.64	1.54	25.17	1931
1932	0.32	1.2	1.03	0.17	4.69	0.04	0.92	5.89	9.95	0.12	1.4	0.68	26.41	1932
1933	0.4	0.61	0.1	0.19	0.29	1.28	0.11	0.49	1.35	1.43	0.33	0.21	6.79	1933
1934	0.67	0.15	0.59	1.17	1.77	0.28	2.23	0.1	3.75	1.83	0.11	0.61	13.26	1934
1935	0.36	0.79	2.34	0.67	4.89	13.71	2.77	0.1	5.86	0.91	0.37	1.85	34.62	1935
1936	0.33	0.01	0.89	1.3	3.72	4.18	1.45	1.2	2.97	1.57	0.55	0.32	18.49	1936
1937	0.21	0.18	0.92	0.18	0.26	0.44	0.88	0.03	0.41	1.49	1.84	3.93	10.77	1937
1938	1.38	0.3	0.35	1.1	0.55	0.6	2.78	0.37	0.36	0.23	T	0.89	8.91	1938
1939	1.46	0.14	0.75	0.52	1.59	1.27	0.68	4.47	1.07	1.12	1.59	1.47	16.13	1939
1940	0.33	1.3	2.21	1.98	3.77	5.13	0.08	5.62	0.18	0.79	0.52	0.38	22.29	1940
1941	0.61	1.13	1.89	1.81	1.21	0.8	3.35	0.95	4.9	7.21	0.22	0.25	24.33	1941
1942	0.3	0.75	0.01	0.99	4.33	0.7	0.13	2.92	3.96	1.58	0.83	0.39	16.89	1942
1943	0.58	0.05	0.57	1.37	4.87	1.28	2.51	0	2.47	1.55	1.72	1.58	18.55	1943
1944	1.85	1.46	0.45	1.05	3.27	2.55	0.01	4.19	3.94	6.5	1.15	1.63	28.05	1944
1945	1.14	0.82	2.53	0.29	0.96	0.4	0.59	0.57	0.9	4.06	0.06	0.06	12.38	1945
1946	1.84	0.5	0.17	3.33	2.83	4.9	0.07	0.27	1.56	1.02	0.1	0.66	17.25	1946
1947	3.05	0.32	1.05	0.64	2.62	2.06	0.41	3.58	1.34	0.69	1.81	0.64	18.21	1947
1948	0.08	0.74	0.11	0.91	1.7	3.56	6.19	0.39	2.15	1.56	0.38	0.15	17.92	1948
1949	2.02	7.82	0.35	4.65	0.4	1.8	0.37	3.29	5.58	1.88	T	2.31	30.47	1949
1950	0.16	0.35	0.07	0.88	2.05	1.47	1.58	0.6	3.5	T	0.04	T	10.7	1950
1951	T	0.07	1.38	0.33	2.15	0.6	T	0.02	1.73	2.09	0.05	0.42	8.84	1951
1952	0.14	0.5	0.38	1.31	3.12	0.26	0.39	0	0.04	T	0.97	0.56	7.67	1952
1953	0.57	0.16	1.7	T	0.2	0.02	T	2.1	4.55	2.58	T	0.4	12.28	1953
1954	0.1	0.01	T	3.66	1.98	4.34	0.4	0.37	1.75	2.01	0.16	T	14.78	1954

TABLE 5-3
MONTHLY/ANNUAL/AVERAGE PRECIPITATION
DEL RIO TX (1905 - 2012)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	YEAR
1955	0.48	0.33	0.71	0.01	2.55	1.02	1.27	1.53	1.21	0.04	1.28	0.23	10.66	1955
1956	0.28	0.34	T	0.05	0.03	0.03	T	0.25	1.07	2.2	0.02	0.07	4.34	1956
1957	0.18	0.94	1.63	6.49	10.23	1.22	T	T	2.76	1.97	1.51	0.82	27.75	1957
1958	3.25	1.67	1.19	0.28	6.17	3.94	0.33	0.88	5.47	4.48	0.24	0.15	28.05	1958
1959	0.08	0.65	T	2.25	3.02	6.65	3.11	0.45	0.69	5.37	0.46	0.72	23.45	1959
1960	0.5	2.17	0.5	0.49	0.51	0.1	4.81	2.52	2.68	3.2	0.33	1.81	19.62	1960
1961	1.56	0.54	0.08	1.3	0.49	7.17	3.51	2.97	0.87	3.21	0.75	0.29	22.74	1961
1962	0.15	0.27	0.17	1.81	0.48	1.11	0.29	0.05	2.1	2.91	0.26	0.31	9.91	1962
1963	T	1.95	T	1.26	2.72	1.49	0.09	T	1.91	0.81	0.63	0.67	11.53	1963
1964	0.42	0.76	0.45	1.2	2.28	0.03	0.34	1.16	15.79	2.47	T	0.25	25.15	1964
1965	0.16	1.72	0.33	3.13	2.34	2.03	0.38	1.64	2.61	0.43	1.26	1.09	17.12	1965
1966	0.94	1.11	0.48	4.36	2.23	3.37	0.05	2.10	1.45	1.26	T	0.04	17.39	1966
1967	0.02	0.29	0.58	1.11	0.65	0.36	0.12	1.00	7.02	1.39	0.66	0.76	13.96	1967
1968	0.66	1.53	1.15	2.53	1.03	1.72	3.07	0.25	1.20	1.84	2.19	T	17.17	1968
1969	1.04	1.17	0.60	5.46	2.36	0.20	1.47	4.27	0.78	11.33	3.36	1.18	33.22	1969
1970	0.42	1.61	0.57	0.14	2.85	1.38	0.04	1.43	9.87	0.06	T	0.12	18.49	1970
1971	T	0.29	T	2.16	0.59	4.87	0.45	6.10	0.50	2.36	0.83	0.25	18.40	1971
1972	0.63	0.08	0.52	1.58	2.61	1.82	2.60	5.74	0.92	0.38	0.71	0.01	17.60	1972
1973	0.76	1.36	1.19	2.08	0.20	3.22	2.43	1.08	2.86	4.78	T	T	19.96	1973
1974	0.05	0.00	1.39	1.39	1.42	T	0.09	3.37	4.98	2.81	0.62	1.47	17.59	1974
1975	0.51	1.92	0.06	1.06	2.17	1.12	8.26	0.27	1.24	1.36	0.10	0.53	18.60	1975
1976	0.15	0.01	0.14	2.45	2.82	0.46	13.18	0.43	1.10	3.20	0.89	1.51	26.34	1976
1977	0.89	1.23	0.87	2.66	2.27	1.23	0.06	0.10	1.41	4.96	0.67	0.01	16.36	1977
1978	0.07	0.36	0.15	2.09	3.46	2.02	2.15	1.14	2.76	0.95	3.35	0.76	19.26	1978
1979	0.29	1.57	2.66	1.53	0.62	4.33	0.25	0.72	0.68	0.00	0.60	0.78	14.03	1979

TABLE 5-3
MONTHLY/ANNUAL/AVERAGE PRECIPITATION
DEL RIO TX (1905 - 2012)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	YEAR
1980	0.22	0.44	0.30	0.44	5.15	0.32	0.28	2.06	1.86	0.06	2.00	1.02	14.15	1980
1981	0.67	0.21	1.64	7.51	1.94	5.54	0.56	2.18	0.36	6.64	0.10	0.06	27.41	1981
1982	0.12	2.45	0.13	0.47	1.59	2.37	0.86	0.12	1.10	0.17	1.77	0.79	11.94	1982
1983	0.70	1.01	1.07	0.13	1.19	1.07	0.32	0.79	1.35	5.09	1.42	0.06	14.20	1983
1984	1.44	0.24	0.06	0.55	2.09	0.45	0.90	0.51	2.78	3.02	1.03	2.44	15.51	1984
1985	1.25	0.51	0.81	2.51	2.20	3.91	1.71	T	2.81	0.63	1.05	0.03	17.42	1985
1986	1.69	0.17	0.02	0.78	3.56	4.47	1.20	0.81	3.70	5.44	0.42	2.36	24.62	1986
1987	0.16	3.47	0.77	3.21	3.89	5.74	1.97	2.05	0.58	0.24	0.45	0.59	23.12	1987
1988	0.05	0.18	0.15	0.12	1.45	1.78	4.38	2.17	4.48	0.27	T	0.38	15.41	1988
1989	1.06	1.04	0.92	0.18	1.94	0.26	0.26	0.23	1.24	0.49	1.32	0.30	9.24	1989
1990	0.59	1.25	3.20	4.82	2.13	T	4.13	0.43	6.19	1.51	1.16	0.22	25.63	1990
1991	0.57	0.75	0.04	1.88	2.37	2.73	1.23	0.75	7.07	0.17	1.42	3.06	22.04	1991
1992	1.92	3.80	2.57	1.27	4.13	1.82	4.28	1.15	0.41	0.80	0.69	0.42	23.26	1992
1993	0.60	0.77	0.47	0.72	1.29	5.28	0.97	2.54	0.77	0.68	0.13	0.18	14.40	1993
1994	2.36	1.61	2.49	1.74	2.61	0.05	5.07	0.57	1.24	1.70	0.39	3.14	22.97	1994
1995	0.03	0.31	1.06	0.75	7.33	0.16	0.08	1.11	3.06	2.13	1.24	0.49	17.75	1995
1996	T	0.39	0.02	0.62	1.20	0.02	0.07	4.77	2.09	0.88	0.87	0.34	11.27	1996
1997	0.20	2.01	2.77	2.55	5.66	3.70	0.77	0.23	1.41	2.39	0.77	0.55	23.01	1997
1998	0.01	0.34	1.06	0.01	0.08	1.35	T	20.93	1.43	1.46	1.94	0.24	28.85	1998
1999	0.01	0.01	1.89	3.17	0.29	5.61	1.48	2.42	0.00	0.39	T	0.01	15.28	1999
2000	0.03	0.94	0.28	0.90	1.03	4.38	0.65	0.11	1.32	5.00	2.82	0.51	17.97	2000
2001	1.08	0.54	0.90	0.22	1.33	T	0.13	0.35	2.24	0.43	1.12	0.35	8.69	2001
2002	0.01	0.02	0.10	1.44	1.81	3.09	0.87	0.63	1.28	7.39	0.73	0.31	17.68	2002
2003	0.32	0.43	0.68	0.09	6.90	1.01	5.34	0.92	3.36	4.47	0.37	0.04	23.93	2003
2004	0.81	0.74	3.48	3.34	2.39	2.28	1.79	2.48	3.96	4.57	4.71	0.40	30.95	2004

TABLE 5-3
MONTHLY/ANNUAL/AVERAGE PRECIPITATION
DEL RIO TX (1905 - 2012)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	YEAR
2005	0.90	1.39	1.74	0.09	2.49	0.10	3.73	1.69	0.02	8.72	T	0.06	20.92	2005
2006	0.25	0.04	0.16	0.59	1.83	2.07	0.01	1.37	2.38	0.53	0.01	0.36	9.59	2006
2007	2.22	0.04	2.36	1.94	7.93	4.61	4.72	1.25	3.49	0.76	1.18	0.32	30.80	2007
2008	0.08	0.02	0.57	0.06	0.58	2.73	0.97	11.32	0.28	0.16	T	0.41	17.21	2008
2009	0.04	0.00	1.53	1.86	0.49	3.06	0.17	0.06	3.37	0.65	0.71	1.02	12.96	2009
2010	2.52	1.54	1.16	6.03	10.45	0.71	4.72	0.57	2.06	0.01	0.01	0.02	29.80	2010
2011	0.08	0.15	0.04	0.01	1.07	0.45	0.37	4.49	1.14	0.39	0.75	0.98	9.92	2011
2012	0.48	1.20	1.31	1.20	4.49	0.01	1.00	0.11	3.90					2012

ALL-TIME:	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL	ALL-TIME:
Maximum	4.12	7.82	3.48	7.51	10.45	13.71	13.18	20.93	15.79	11.69	4.71	3.93	37.75	Maximum
Average	0.67	0.85	0.92	1.63	2.50	2.22	1.88	1.79	2.50	2.28	1.06	0.72	18.52	Average
Median	0.42	0.50	0.64	1.27	2.05	1.51	1.22	0.92	1.73	1.53	0.71	0.42	17.84	Median
Minimum	TRACE	0.00	TRACE	TRACE	0.03	TRACE	TRACE	0.00	TRACE	0.00	0.00	TRACE	4.34	Minimum

6.0 HYDRAULIC ANALYSIS

6.1 Introduction and Software

The floodplain hydraulics for the Master Watershed Study was analyzed using the computer program HEC-RAS, River Analysis System, developed by the Hydrologic Engineering Center of the U.S. Army Corps of engineers (Version 4.1.0, January 2010). The floodplains were analyzed with two (2) plans that include the existing and proposed improvement alternatives. The existing plan incorporates the existing stream conditions, surveyed cross sections, and existing bridge and culvert structures. The proposed alternative plan will modify the existing plan to reflect alternative proposed improvements where appropriate, such as regional detention facilities or channel improvements. Additional plans may be added to reflect combinations of proposed improvements.

6.2 Stream Reaches and Confluences

Part of the layout procedure in HEC-RAS is defining reaches and confluences in the stream. Often a stream reach is defined by flow changes brought on by the addition of tributaries at a confluence or the presence of a flow bypass. In the Master Watershed Study, major stream systems were kept separate as much as possible in an effort to follow established FEMA procedures. FEMA requires tributaries and mainstem streams to have independent boundary conditions. As such, Calaveras Creek, Calaveras Creek Tributary 1, and San Felipe Creek each have a separate hydraulic model. Similarly in the Cienegas Creek watershed, Cantu Branch and the Cantu Branch Tributaries have separate HEC-RAS hydraulic models.

San Felipe Creek is divided into two separate reaches – the Upper_SF and the Lower_SF. Because of overflows at Veterans Boulevard that connect Cantu Branch Tributary 4 to the mainstem, CBT4 is modeled concurrently with the Cantu Branch. All of the other hydraulic models contain a single stream reach.

6.3 Cross Sections

Geometric cross sections were developed to represent the floodplain characteristics of each creek modeled in the San Felipe and Cienegas Creek watersheds. Each cross section spans both the channel and entire floodplain of the respective creek. The cross sections are spaced usually no more than 500 ft apart and are orientated perpendicular to elevation contours and stream centerline. Elevation data for cross sections were extracted from the LIDAR provided by IBWC and supplemented at structures and

selected natural cross sections with field survey data provided by the surveyor (Bain Medina Bain, Inc).

6.4 Manning's "n" Values

Composite Manning's "n" values (roughness coefficients) for overbanks and channel were estimated from aerial photography and field reconnaissance. The overbanks consist primarily of open field or pasture, commercial businesses and single family residences with some brushy or lightly forested areas near the channel. The "n" values generally range from 0.022 to 0.150 along the overbanks and 0.020 to 0.045 in the channel. Areas with "n" values between 0.020 and 0.040 represent paved areas such as streets and parking lots to bare soil with sparse grass and shrubs. Values of 0.045 to 0.150 represent more dense grassy areas to structures such as houses and fences. The blocked obstruction feature in HEC-RAS was used to represent structures (houses, buildings, fences) that could potentially impede flow in the overbank areas.

6.5 Boundary Conditions

The downstream boundary condition was applied using the normal depth method with slopes ranging from 0.0035 ft/ft for San Felipe Creek to 0.0086 ft/ft for Cantu Branch Tributary 4. Because of the flat grades of the channel invert, the sub-critical flow regime in HEC-RAS was employed for this hydraulic analysis. Flow change locations in the HEC-RAS model correspond with the HEC-HMS sub basin combination points located at specified design points. The flow changes reflect the Modified Puls Routing established along each creek.

6.6 Ineffective Flow Areas

Ineffective flow areas have been introduced upstream and downstream sides of structures located on the creeks and tributaries. Ineffective flow contraction ratio of 1:1 (H:V) and an expansion ratio of 4:1 were used for the upstream and downstream side of the structures, respectively.

6.7 Expansion / Contraction Coefficients

In the development of the hydraulic models for Cienegas and San Felipe Creeks, expansion and contraction coefficients consistent with those detailed in the HEC-RAS Hydraulic Reference Manual have been employed in the Del Rio Study. Typical geometric cross sections use the default values of 0.1 and 0.3. For bridges such as U.S. 90 over San Felipe Creek where the roadway embankment encroaches the floodplain width somewhat, values of 0.3 and 0.5 are used.

6.8 Split Flow Analyses

At the upper end of the Cantu Branch of Cienegas Creek, split flow occurs where a portion of the flow "short-circuits" the Cantu Branch behind or to the east of the Regency Inn on U.S. 377 and travels south through open field before joining Danielle Street and joining the Cantu Branch north of the Wal-Mart near U.S. 377 / Veterans.

This split flow bypass of the upper Cantu Branch was modeled by simulating the overland flow portion of the bypass with an inline weir. All of the flow that flowed over the weir and consequently left the channel, was removed from the flow that continued downstream in the Cantu Branch. In order to conserve the flow generated by the storm, a similar magnitude of flow was added back into the Cantu Branch north of the Wal-Mart. Several iterations of flows changes were made until the water surface elevations balanced on the north side of the neighborhood near Lausen Lane.

6.9 Structures / Crossings

There are twenty-three (23) bridges and box/pipe culverts, 7 low water crossings or at-grade crossings, and two (2) dams or weirs within the City limits of Del Rio. There are also 23 sites requiring barricades before each storm due to localized flooding mainly in city streets. Table 6-1 lists the structures located within the City Limits of Del Rio:

Table 6-1: Existing Structure Locations and Type

Watershed	Creek/Tributary	Street	Type	Size/Span
San Felipe	**San Felipe**	US HWY 90	Double Opening Bridge	5 - 30'/7 - 30'
		RR (Below US 90)	Double Opening Bridge	
		Bedell Ave	Bridge	1 - 70'
		US HWY 277	Bridge	11 - 30'
		Gillis/Losoya St	Bridge	5 - 30'
		Johnson St.	Bridge	1 - 28'
		E. Canal St.	Bridge	9 - 30'
		E. Academy St.	Bridge	1 - 42'
		Near Johnson St	Tardy Dam	155'
		Near Magnolia	In-line Weir	65'
	Calavaras	US HWY 90	Box Culvert (east)	5 – 6′ x 6′
		US HWY 90	Box Culvert (west)	6 – 6′ x 6′
		Rail Road	Span Culvert	2 – 15′ x 14′
		Railway Ave	Pipe	1 – 4′ x 3′
		US HWY 277	Box Culvert	5 – 12′ x 4′
		Broadbent Ave	Bridge	3 - 17'
	Calaveras Trib 1 (CVCT1)	Calderon Blvd	Box Culvert	3 – 7′ x 5′
		Plaza Ave	Pipe	1 - 3'
		W. Bowie St.	Pipe	1 - 5'
Cienegas	**Cantu Branch**	US HWY 90	Box Culvert	9 – 7′ x 4′
		Amistad Blvd	Pipe	7 - 24"
		Margaret	Pipe	10 - 24"
		King's Way	Low Water Crossing	N/A
		Dodson St	Low Water Crossing	N/A
		Chapman Rd	Low Water Crossing	N/A

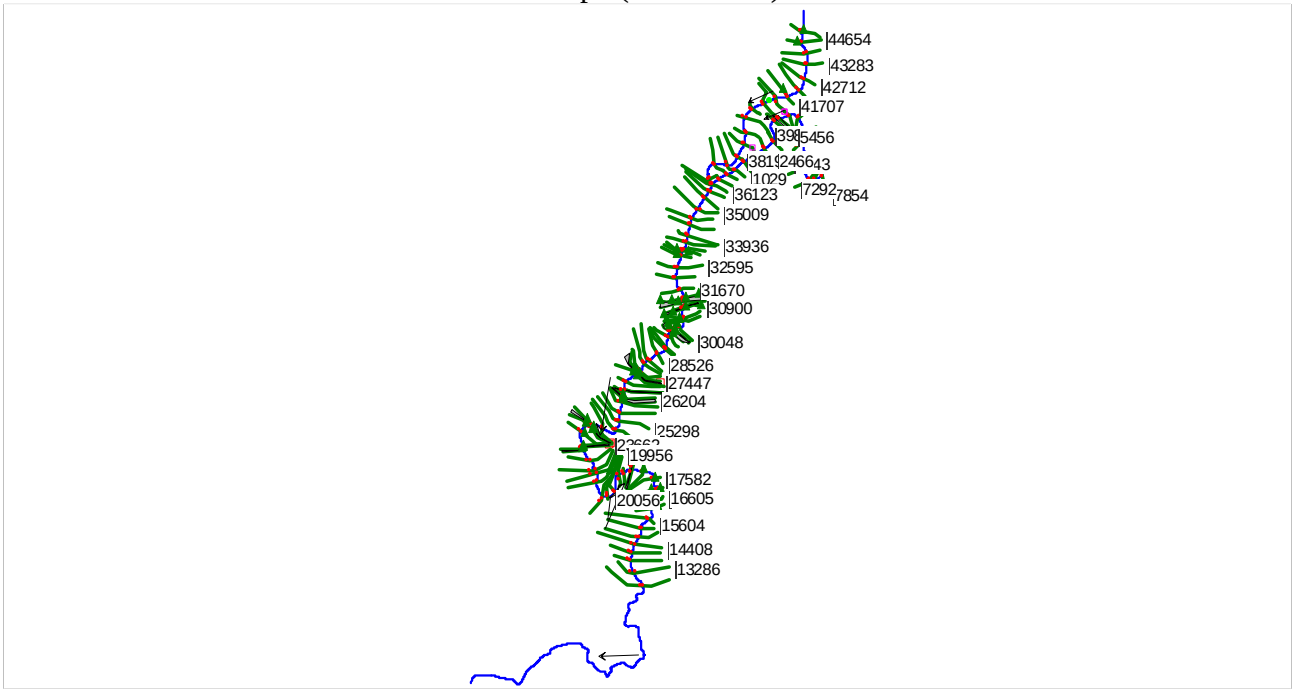
Table 6-1: Existing Structural Location (cont'd)				
Watershed	**Creek/Tributary**	**Street**	**Type**	**Size/Span**
	Cantu Branch Trib 1	King's Way	Low Water Crossing	N/A
		Amistad Blvd	Low Water Crossing	N/A
		King's Way	Low Water Crossing	N/A
	Cantu Branch Trib 4	US HWY 90	Box Culvert	6 – 6′ x 3′
	Cantu Branch Trib 5	US HWY 90	Box Culvert	2 – 5′ x 6′
	Cienegas Trib 1	Wildcat Dr	Pipe	2 – 36″

Descriptions of the existing level of service for each structure and the associated overtopping depths will be discussed in Chapter 7 – Flood Control.

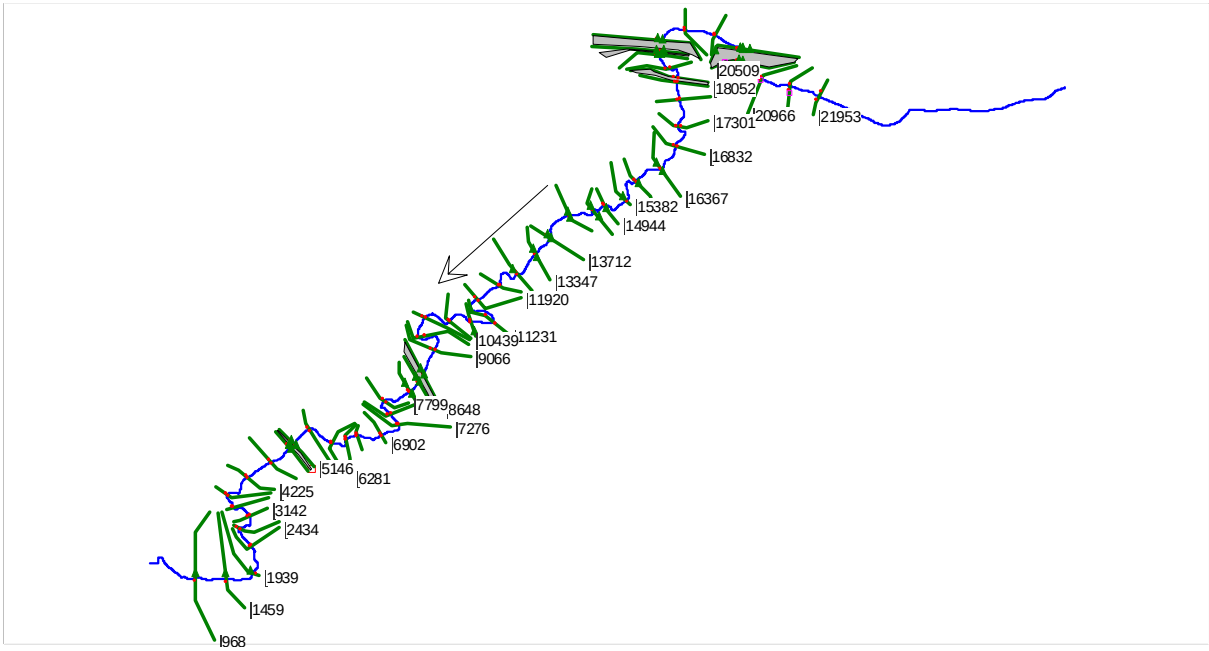
6.10 HEC-RAS Section Locations

Besides the flows generated by the hydrologic model, and the connectivity of the reaches and streams, the main component of the hydraulic model is the geometric cross section and its associated channel roughness.

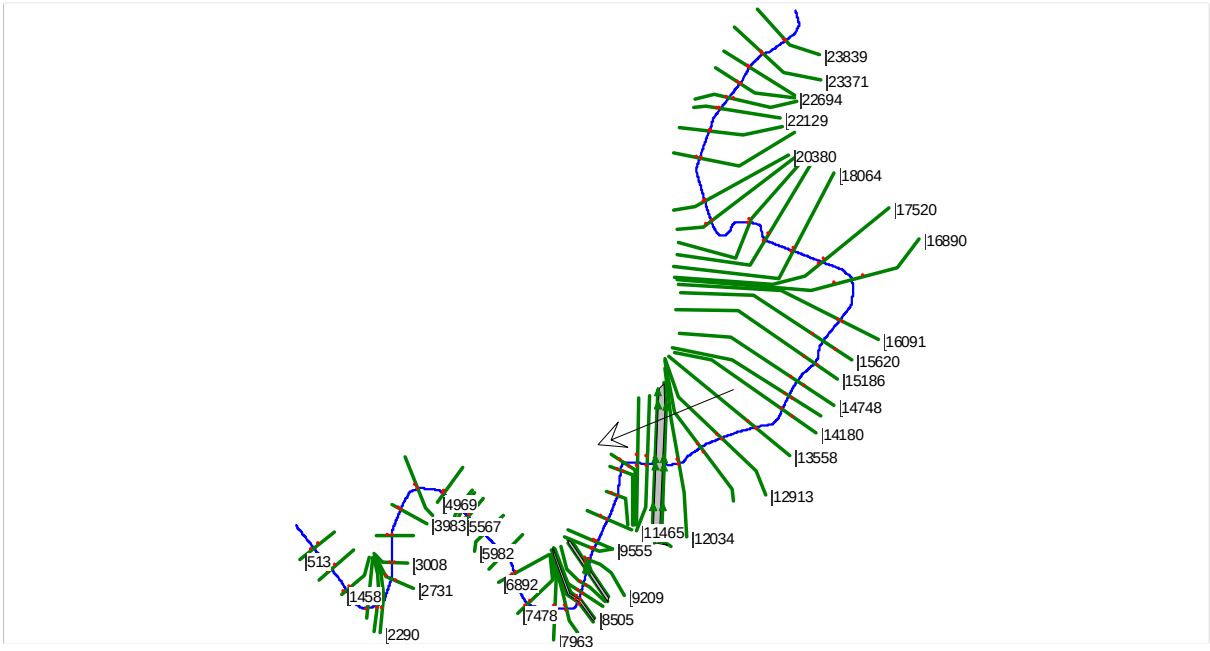
San Felipe (Main Stem)



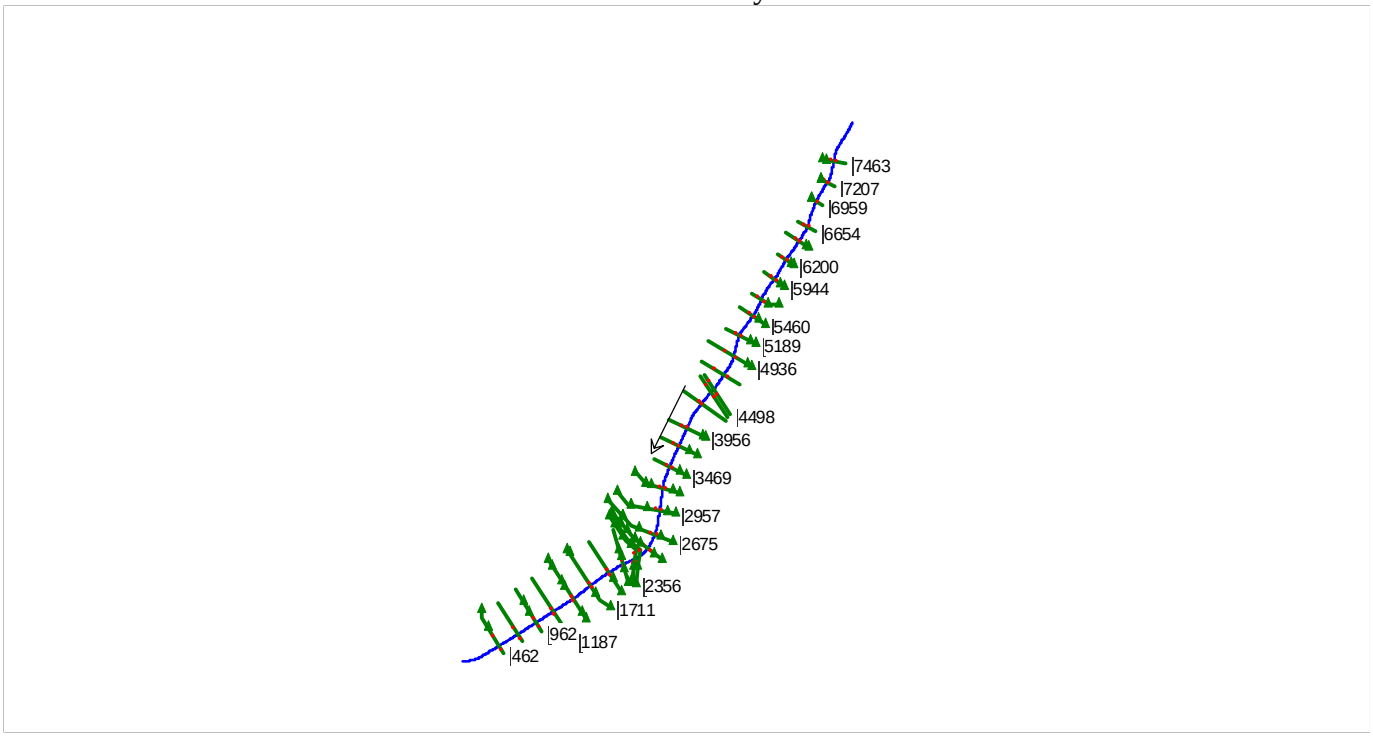
Calaveras Creek



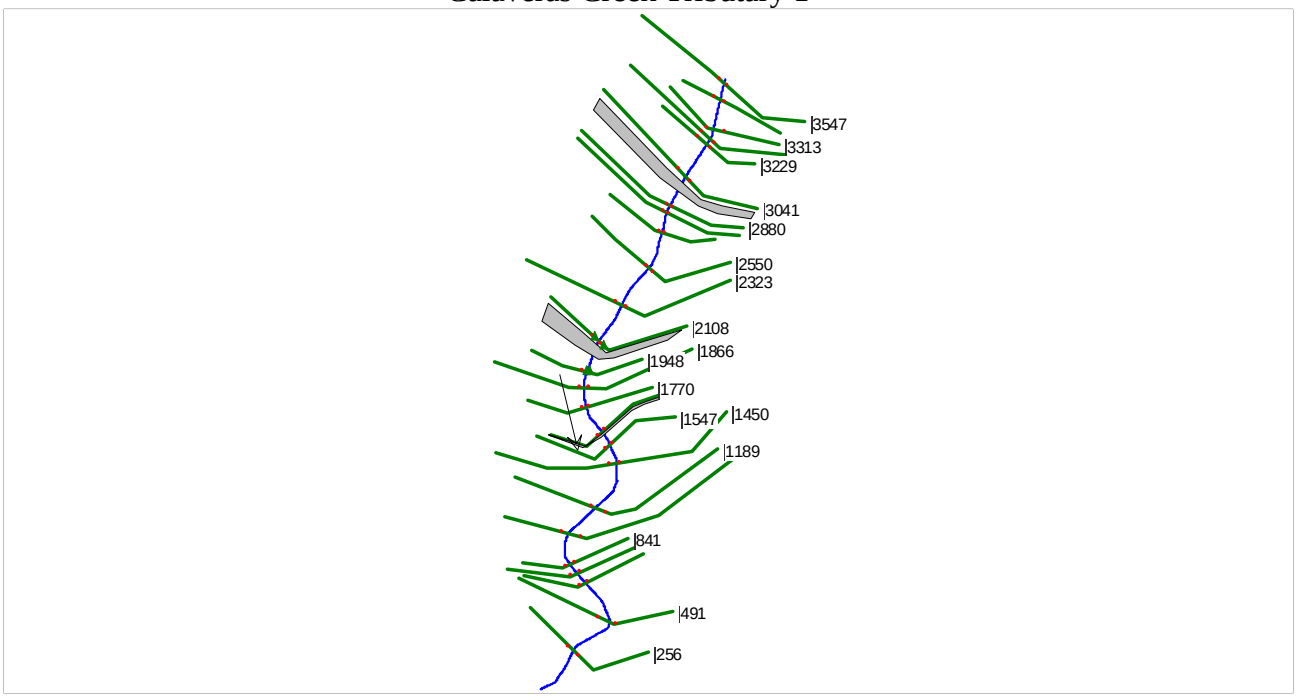
Cantu Branch



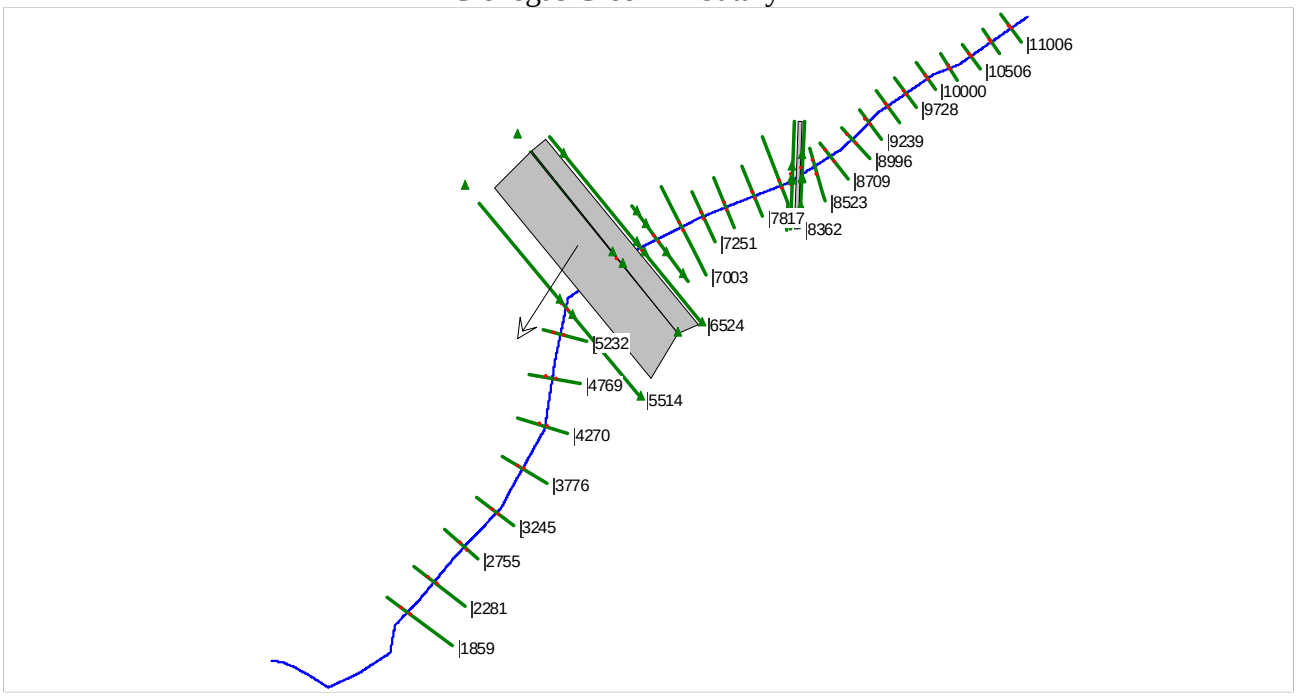
Cantu Branch Tributary 1



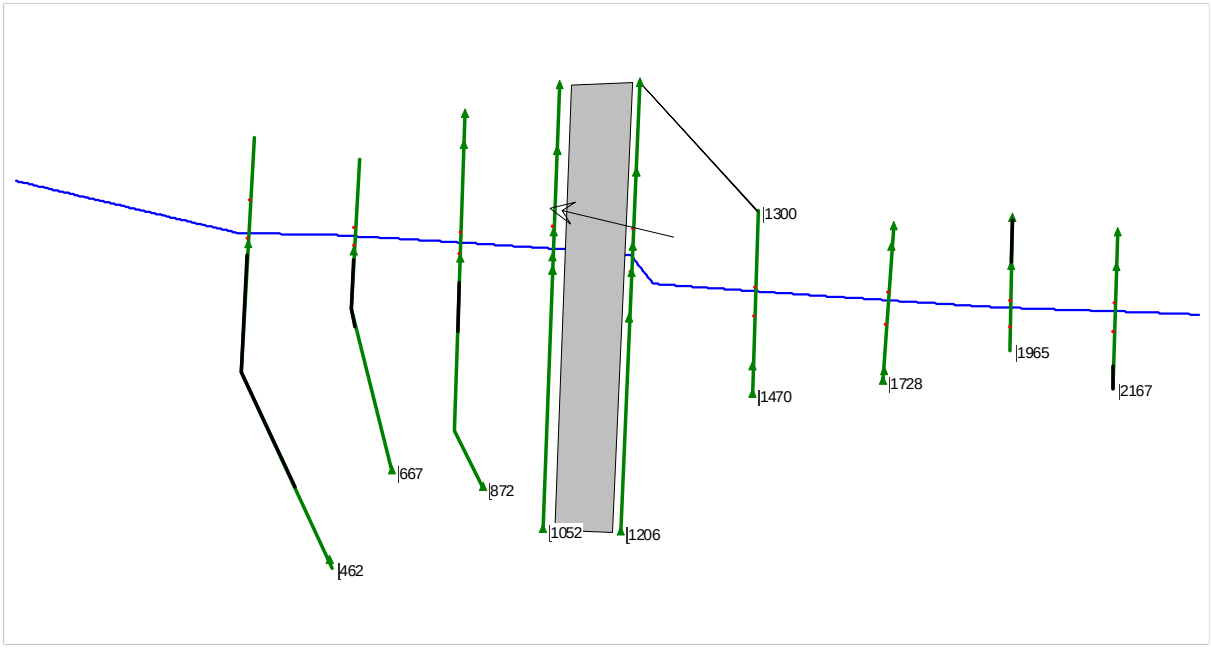
Calaveras Creek Tributary 1



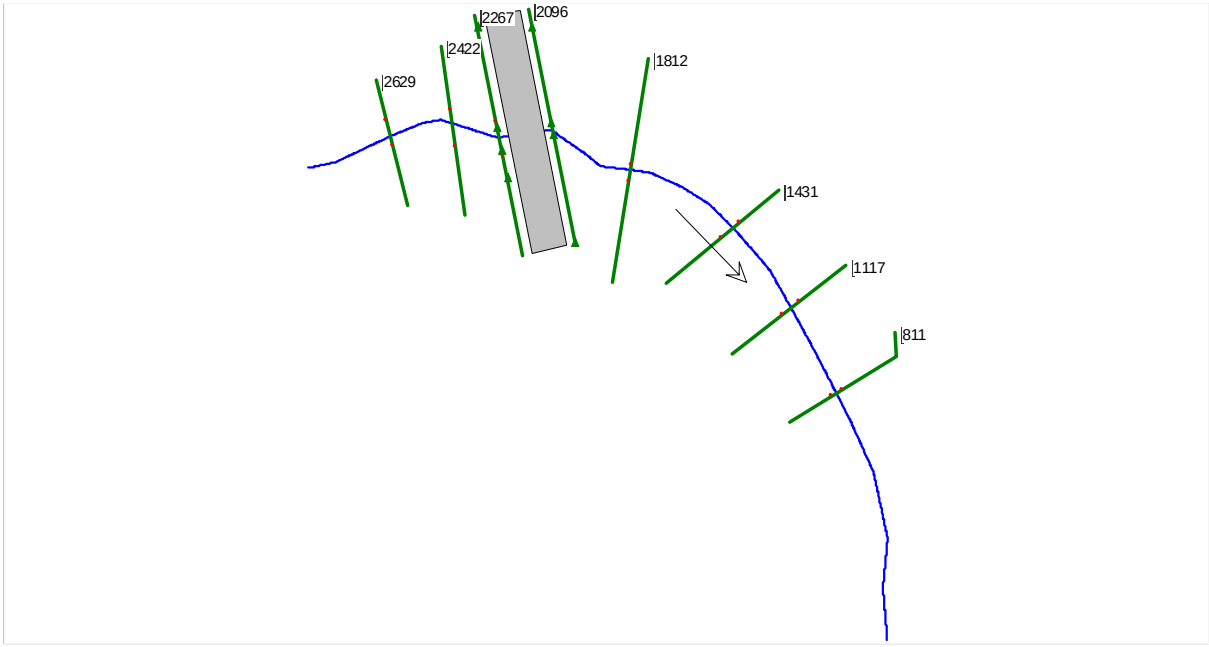
Cienegas Creek Tributary 1



Cantu Branch Tributary 4



Cantu Branch Tributary 5



References:

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7.0 FLOOD CONTROL – IDENTIFICATION OF AREAS OF CONCERN

7.1 Introduction

The studied reaches in the San Felipe Creek watershed consists of Calaveras Creek, Calaveras Creek Tributary 1, San Felipe Creek Tributary 1 and the San Felipe Creek mainstem. There are several areas adjacent to San Felipe Creek that are prone to flooding, mainly in the residential areas along the creek. The locations of primary concern for potential residential flooding during the 100-year flood event include areas downstream of US HWY 90, primarily due to backwater upstream of bridge and culvert structures. Studied reaches in the Cienegas Creek watershed include (from east to west) Cienegas Creek Tributary 1, Cantu Branch, Cantu Branch Tributary 1, Cantu Branch Tributary 4, and Cantu Branch Tributary 5.

Table 7-1 summarizes the flood water elevations for the 2-, 10-, and 100-yr storm events at the bridge and culvert structures and low water crossings in the Master Watershed Study Area by watershed.

Table 7-2 lists the current level of service of the roadway crossings and bridges. It also provides the depth of water which overtops the roadway for the 100-yr storm event. The statistics from the national website for "Turn Around, Don't Drown" indicate that six inches of fast moving water will knock a person down and two feet of water at a crossing will move even large vehicles downstream. By this yardstick, nearly all of the crossings are dangerous to the walking public during an extreme event, and the majority of the San Felipe and Cantu Branch crossings are dangerous to vehicular traffic. The proposed improvements listed in Chapter 9 will not provide 100-yr protection to every structure, but they will decrease the probability that the roadways will be inundated from almost every storm event to less than 10% of the time.

7.2 San Felipe Creek Watershed

7.2.1 Calaveras Creek

Calaveras Creek has residential structures only to the north and west. There is very minimal inundation to structures along the creek. The area just downstream of US HWY 277 adjacent to Leticia Street has a few residential structures within the 100-year floodplain boundary. The majority of the flooding is confined to the south and east of the creek where there is little or no development currently.

Roadway crossings where frequent flooding is a public safety issue include Railroad Avenue.

7.2.2 Calaveras Creek Tributary 1

The Calaveras Creek Tributary 1 flows through heavily residential area to the north of Calaveras Creek. Residential structures between Plaza Avenue and Bowie Street are inundated by the 100-year flood due to backwater at the undersized Bowie Street culvert. Commercial structures upstream of Plaza Avenue are also inundated by the 100-year floodplain.

Roadway crossings where frequent flooding is a public safety issue include the intersection at Plaza and W. Viesca Street, and Bowie Street.

7.2.3 San Felipe Creek

The Gillis Street bridge crossing at San Felipe Creek has significant flooding immediately upstream of the bridge, inundating several residential structures between McLymont Street and De La Rosa Street to the south and structures along Losoya/Gillis Street to the north. The bridge structure and roadway are overtopped by 50-year or greater storms and has an approximate 100-year floodplain width of 1,500 feet upstream of Gillis Street.

The Johnson Street bridge crossing has significant flooding immediately upstream and downstream of the bridge, inundating several residential structures between Broadbent Street and Cortinas to the south. The bridge structure and road are overtopped by 2-year or greater storms and has an approximate 100-year floodplain width of approximately 915 feet upstream of Johnson Street.

The E. Canal Street bridge crossing at San Felipe Creek has significant flooding immediately upstream and downstream of the bridge, inundating several residential and commercial structures between Flores Street and Cantu Street to the south. Flooding in this area is due to backwater from Canal Street bridge and meandering and bends in the creek. The bridge structure and road are overtopped by 25-year or greater storms and has an approximate 100-year floodplain width of approximately 1,715 feet upstream in the vicinity of Canal Street.

7.2.4 San Felipe Creek Tributary 1

The flooding issues in the San Felipe Creek Tributary 1 are associated with shallow neighborhood flooding as opposed to riverine flooding in a creek or stream. Runoff from the upper watershed near W. 13th flows through the neighborhoods until it backs up along 10th Street before draining south along Airport.

7.3 Cienegas Creek Watershed

7.3.1 Cienegas Creek Tributary 1

The 100-year flood event has no significant impacts on any structures, residential or commercial. Wildcat Drive is inundated by the storms smaller than the 2-yr storm event.

7.3.2 Cantu Branch

There are several areas along Cantu Branch that are prone to significant flooding.

King's Way near Mary Lou has significant flooding because the roadway crossing is an "at grade" crossing. There are flashing lights to warn of potential flooding across the roadway, but any barricades must be placed by hand.

Amistad Boulevard downstream through the Margaret Lane low water crossing has significant flooding inundating many residential and commercial structures. It is likely due to the meandering or bending of the creek in the area.

US 277/90 Hwy crossing has significant flooding potential downstream and upstream. The road is overtopped by the 50-year event or greater and several commercial properties are inundated during the 100-year event. North of Wal-Mart on the upstream side of Hwy 277 there is a large open field which is almost entirely inundated by the 100-year flood boundary. This area accounts for approximately 400 ac ft of valley storage.

The neighborhood upstream of Lausen Lane has had flooding in the past, both from Cantu Branch to the east encroaching into the neighborhood, but also by flood flows bypassing the channel and traveling down Danielle Drive. During the May 2010 storm, pavement was removed by fast moving waters at the northern end of Danielle Drive.

7.3.3 Cantu Branch Tributary 1

The 100-year flood boundary is contained almost entirely in the creek, and few residential structures show to be inundated. There are duplexes or multifamily units at the corner of King's Way and Arrowhead Trail that may become inundated during the 100-year flood.

King's Way at the CBT1 is the northernmost portion of King's Way to be overtopped during storm events. There are no culverts under the existing roadway crossing – it is

an at-grade crossing. Even though it is higher up in the watershed than the other King's Way crossings, it can still be a potential problem site if flows get too deep.

7.4 Results – Summary of Water Surface Elevation

Water surface elevations at major road crossings are shown in Table 7-1 below.

TABLE 7-1
WATER SURFACE ELEVATIONS AT STRUCTURES

Structure Crossing	River ID	Cross Section ID	WSEL (ft)		
			2-Yr	10-Yr	100-Yr
US HWY 90	SF	**31290**	956.85	959.35	965.93
RR (Below US 90)	SF	**30947**	954.15	957.36	964.72
Bedell Ave	SF	**29758**	950.25	955.39	961.16
US HWY 277	SF	**29721**	949.94	955.10	960.97
Gillis/Losoya St	SF	**27447**	946.02	950.20	953.38
Johnson St	SF	**26277**	939.08	944.32	947.37
Canal St	SF	**23794**	932.57	936.74	940.19
E Academy St	SF	**22671**	927.11	931.85	936.89
Near Magnolia	SF	**18657**	919.67	923.14	925.69
US HWY 90 (East)	CVC	**20509**	1003.59	1006.07	1008.58
US HWY 90 (West)	CVC	**18906**	992.89	995.56	1001.01
Rail Road	CVC	**18722**	992.51	994.08	997.42
US HWY 277	CVC	**8648**	945.06	950.77	952.13
Broadbent Ave	CVC	**5146**	931.20	934.43	936.08
Calderon Blvd	CVCT1	**3041**	954.82	956.44	959.01
Plaza Ave	CVCT1`	**2108**	950.39	950.82	951.23
W. Bowie St	CVCT1	**1623**	946.12	946.46	946.65
Lausen Rd	CB	**14748**	1035.35	1035.98	1036.32
US HWY 90	CB	**11767**	1030.82	1033.72	1035.85
Amistad Blvd	CB	**9095**	1023.12	1023.97	1026.33
Margaret	CB	**8505**	1022.43	1023.52	1025.32
Alta Vista Dr	CB	**5402**	1011.96	1013.08	1016.09
King's Way	CB	**2290**	1000.95	1002.26	1005.23
King's Way (North)	CBT1	**6654**	1064.12	1064.41	1064.69
Amistad Blvd	CBT1	**4498**	1029.56	1029.75	1029.98
King's Way (South)	CBT1	**2356**	1010.19	1010.77	1011.43
US HWY 90	CBT4	**1206**	1037.20	1037.69	1038.21
US HWY 90	CBT5	**2267**	1067.02	1069.64	1070.27

Note: WSEL taken from cross section upstream of structure.

Table 7-2 Existing Level of Service and 100-yr Overtopping Depth

Watershed	Creek/Tributary	Street	Level of Service	100-yr Overtopping Depth (ft)
San Felipe	**Calavaras**	US HWY 90 (east)	10-yr	0.77
		US HWY 90 (west)	10-yr	1.8
		Railroad	100-yr	N/A
		Railway Ave	<2-yr	2.5
		US HWY 277	5-yr	1.9
		Broadbent Ave	5-yr	1.8
	Calaveras Trib 1 (CVCT1)	Calderon Blvd	50-yr	<1
		Plaza Ave	<2-yr	2
		W. Bowie St	<2-yr	1.2
	San Felipe	US HWY 90	10-yr	3.9
		RR (Below US 90)	100-yr	N/A
		Bedell Ave	<2-yr	16.0
		US HWY 277	50-yr	0.93
		Gillis/Losoya St	5-yr	6.5
		Johnson St	<2-yr	15.4
		E. Canal St	10-yr	3.5
		E. Academy St	<2-yr	16.0
Cienegas	**Cantu Branch**	US HWY 90	5-Yr	1.5
		Amistad Blvd	< 2-Yr	6.2
				5.8
		Margaret	< 2-Yr	
		King's Way	< 2-Yr	7.8
		Dodson St		N/A
		Chapman Rd		N/A
	Cantu Branch Trib 1	King's Way	< 2-Yr	3.3
		Amistad Blvd	< 2-Yr	0.97
		King's Way	< 2-Yr	1.6

Table 7-2 Existing Level of Service and 100-yr Overtopping Depth (cont'd)

Watershed	Creek/Tributary	Street	Level of Service	100-yr Overtopping Depth (ft)
Cienegas	**Cantu Branch Trib 4**	US HWY 90	25-Yr	Mixes with adjacent basin
	Cantu Branch Trib 5	US HWY 90	2-Yr	1.2
	Cienegas Trib 1	Wildcat Dr	<2-yr	1.3

Table 7-3 includes a summary of the flooding impacts for the existing conditions 100-yr event. The impacts were determined using GIS tools by identifying the parcels that are fully or partially within the 100-year floodplain.

Table 7-3 Existing 100-yr Flood Impacts

Watershed	Creek/Tributary	Existing Conditions - Number of Parcels Impacted
San Felipe	**San Felipe**	941
	Calavaras	45
	Calaveras Trib 1	80
Cienegas	**Cantu Branch**	201
	Cantu Branch Trib 1	13
	Cantu Branch Trib 4	0
	Cantu Branch Trib 5	0
	Cienegas Trib 1	2

8.0 ENVIRONMENTAL PERMITTING

8.1 Environmental and Cultural Resource Report

As part of the Master Watershed Study, CP&Y also prepared an environmental and cultural resource report. This report looked at the locations of all of the proposed problem areas where CIPs might be constructed so as to gauge any potential environmental permitting roadblocks.

The Master Watershed Study was developed in order to identify and prioritize recurring drainage problem areas. This study will lead to the recommendation of specific drainage infrastructure improvements that are shown to have significant benefits in comparison to cost. It will also aid in providing the City with pertinent information to support or reject proposed actions. The eight study areas identified for this resource review are located throughout the City of Del Rio in Val Verde County, Texas. These study areas are detailed in **Table 8-1** below and illustrated in **Figure 8-1**.

Study Area	Description	Location	Area (acres)
A	Cantu Branch and the Tributary of Cantu Branch (CBT4)	From the intersection of Encino Drive and Cantu Branch meandering upstream until just south of Qualia Road; From approximately 0.3 miles west of Kings Way upstream northeast of Kings Way (also known as FEMA Stream 3)	1,381
B	Tributary of San Felipe Creek (SFCT1)	From the confluence with San Felipe Creek upstream to the northern end of Riverview Drive	416
C	Tributary to Cienegas Creek (CNCT1)	From the confluence with Cienegas Creek upstream through the Del Rio International Airport to near Main Street (also known as FEMA Stream 2)	156
D	Del Rio Airport Area	Drainage to Cienegas Creek near the Airport from 15th Street to West Gibbs	237
E	Tributary to Cienegas Creek (CNCT1)	From the confluence with Cienegas Creek upstream through the Del Rio International Airport to near Main Street (also known as FEMA Stream 2)	144
F	San Felipe Creek	From the confluence with the Rio Grande floodplain upstream to the northern end of the San Felipe Country Club	364
G	Tributary to Calaveras Creek (CVCT1)	From the confluence with the Calaveras Creek mainstem upstream to US 277 (also known as FEMA Stream 1)	207
H	Calaveras Creek	At the bend in Calavaras Creek near Railway Avenue and US 90	6
		TOTAL	**2,911**

The entire Environmental and Cultural Resource Report is included in Volume II of the Master Watershed Report. It is included as **Appendix E.**

The concluding paragraphs of the Environmental and Cultural Resource Report are included here.

Section 404 of the Clean Water Act (CWA), the Antiquities Code of Texas, and the Endangered Species Act apply to projects funded by the City of Del Rio. No jurisdictional wetlands were identified during the July 2012 field event. OHWMs were recorded for waters of the U.S. identified within the study areas. There is a high potential for archeological resources in the study areas due to the presence of water sources and previously documented sites throughout Del Rio. There is a moderate potential for the presence of historic architectural resources within the study areas; specifically, features associated with the Canal System of Del Rio and San Felipe Springs. It is assumed that no federally-listed species will be adversely affected by the proposed project. Habitat was identified for two of the federally-listed species in Val Verde County: the Devil's River Minnow and Texas hornshell. However, no work will be completed within San Felipe Creek, which is the only waterbody in the study areas that provides habitat for these two species.

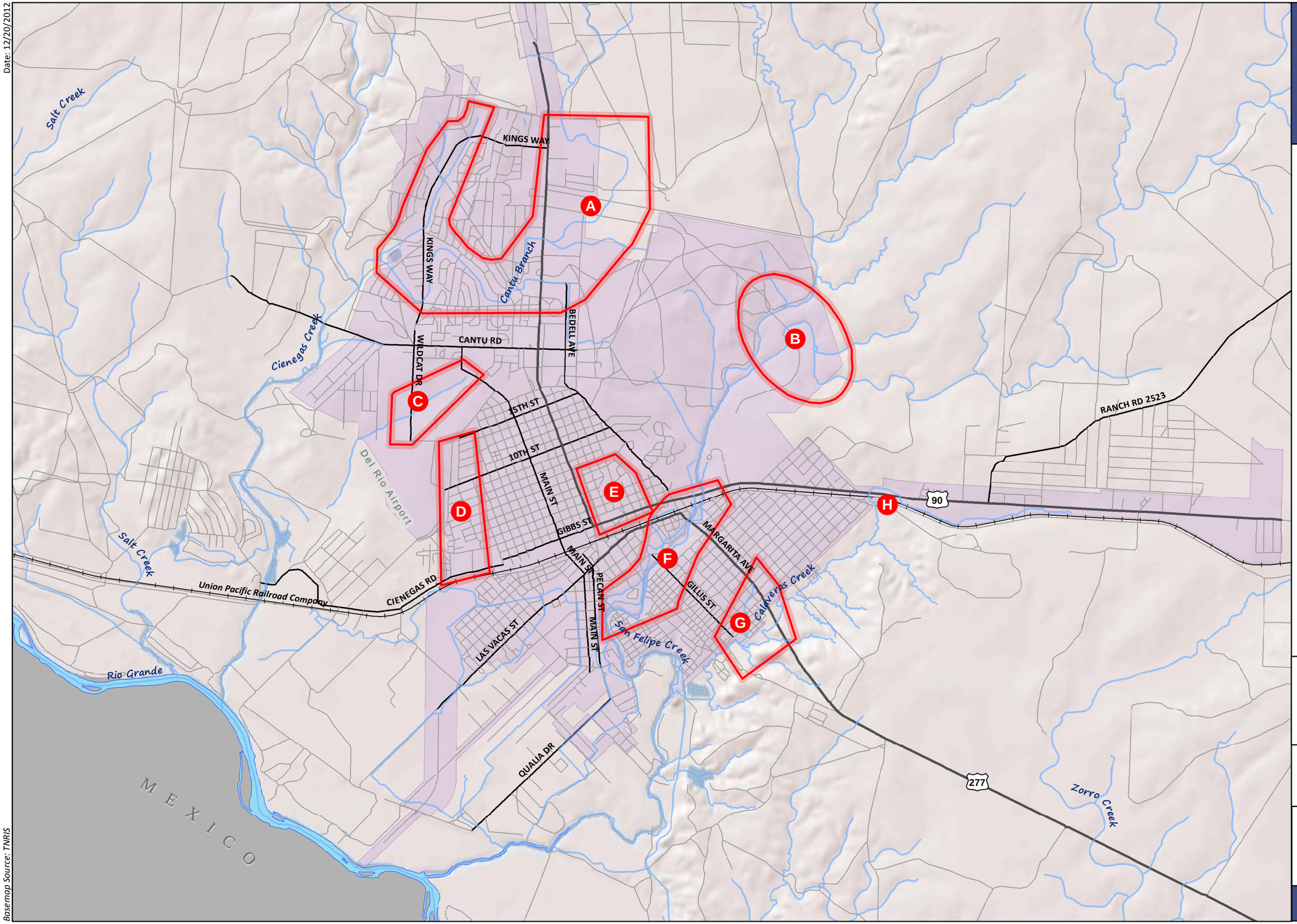
Various flood control solutions are being recommended to the City for a Capital Improvements Program. When the engineers and environmental/cultural staff were discussing the flood control alternatives, numerous Nationwide Permits (NWP) were reviewed for relevance to the proposed options. Since a detailed design is not currently available, it is difficult to state exactly what permit would be required. It is likely that the proposed options would fall under one or more of the following Nationwide Permits (NWP):

- NWP 3 – Maintenance
- NWP 13 – Bank Stabilization
- NWP 14 – Linear Transportation Projects
- NWP 18 – Minor Discharges
- NWP 27 – Aquatic Habitat Restoration, Establishment, and Enhancement Activities
- NWP 41 – Reshaping Existing Drainage Ditches
- NWP 43 – Stormwater Management Facilities
- NWP 46 – Discharges in Ditches

It is also likely that the proposed flood control work can be completed under a NWP rather than an Individual Permit (IP), assuming that no work will be completed in the area of Devil's River minnow critical habitat in San Felipe Creek. If federal funds are utilized for the proposed project, a National Environmental Policy Act (NEPA) document will likely be required. The level of document will be determined on the type of work performed and the funding agency. When the City has selected an option and a detailed

design has been developed, the environmental permitting requirements should be reassessed. While additional biological field work might be required, an archeological survey would be required.

Date: 12/20/2012



Basemap Source: TNRIS

Project Location and Study Areas

FIGURE 8-1

9.0 FLOOD CONTROL SOLUTIONS

There are several types of solutions available to the City: channel improvements to lower water levels and shrink the floodplains, bridge or culvert improvements to raise roadways out of the floodplain, regional detention facilities to reduce flood flows, and buy-outs to remove structures from the floodplain. All of these solutions have their place in the City.

Proposed project sites were chosen based on many discussions with City staff and multiple site visits. Some types of projects were not considered because of conflicts with current City goals such as the San Felipe Creek Master Plan. Over 20 flood control projects are presented in this section of the report. These projects encompass sites in each watershed in the City limits. The estimated construction costs range from $125,000 to over $18.5 million. Table 9.1 summarizes the projects considered by watershed, moving from east to west across the City. This is not the recommended order or a prioritized list.

TABLE 9-1 LIST OF PROJECTS BY WATERSHED

Project ID	Description	Est. Cost
Calaveras Creek		
Railroad Avenue	Road/Culv Imp.	$198,000
Plaza/W.Viesca St	Road/Culv Imp.	$144,000
Bowie Street	Road/Culv Imp.	$124,000
Calaveras Creek Tributary 1		
Vitela Street	Off Channel Detent.	$345,000
San Felipe Creek		
RSWF A	Regional Detention	$3,156,000
RSWF B	Regional Detention	$18,514,000
Johnson Street	Bridge Replacement	$414,000
Canal Street	Bridge Replacement	$994,000
Academy Street	Bridge Replacement	$354,000
San Felipe Creek Tributary 1		
W. 10th Street	Stormsewer	$1,136,000
Cienegas Creek Tributary 1		
Wildcat Drive	Road/Culv. Imp.	$182,000
Cantu Branch		
Kings Way– Site 1	Road/Culv Imp.	$299,000
Margaret Lane	Road/Culv Imp.	$120,000
Amistad Blvd	Road/Culv Imp.	$226,000
Cantu Branch Trib 1		
Kings Way– Site 2	Road/Culv Imp.	$289,000
Kings Way– Site 3	Road/Culv Imp.	$194,000
Amistad– Site 2a	Road/Culv Imp.	$448,000
Amistad– Site 2b	Road/Culv Imp.	$226,000
Kings Way/Amistad Blvd	Stormsewer Imp.	$336,000

Detailed opinions of construction costs are included in **Tables B 9-1** through **B 9-19** in **Appendix B** in Volume II of the report. While every effort is made to include all of the major items in the cost opinions, these should not be considered ready for bid. The cost opinions are more applicable for initial budgeting and cost comparisons. Land acquisition costs are a major variable in the cost estimates of some of the proposed projects. Finally, the amount of land required for the regional stormwater facilities may be less than shown in the cost spreadsheet. The land acquisition portion of the spreadsheets represents the most conservative estimate of easements or land to be impacted.

9.1 Calaveras Creek Solutions

9.1.1 Regional Stormwater Facilities
Trib 1 off-channel detention

There is an empty lot on the west side of the Tributary 1 channel that could be made available as an off-channel detention area for flood control mitigation or as a potential site for water quality treatment in the future. The size of the contributing watershed is quite large with respect to the pond site, so the design of the structure to capture just the peak of the storm runoff would be most beneficial. There are three residential structures nearby that would need to be considered during any detailed designs.

9.1.2 Roadway Improvements

Plaza & W. Viesca Street Culvert Improvements

There is an undersized culvert which drains Calaveras Creek Tributary 1 from the north to the south. One (1) 24-in CMP allows low flows to continue downstream. Most storm events overwhelm the single culvert and overtop the intersection, leading to pavement failure. It is proposed to replace the single CMP with two (2) 6'x4' MBC. This would provide a 2-yr protection. The roadway would remain at the same vertical profile. The estimated cost would be $144,000.

Railroad / Landfill Access

Railway Avenue is inundated once the existing 48" RCP is at capacity. The proposed solution for Railway Avenue is to replace the low water crossing with 2 – 6'x5' MBC. This would provide a 2-yr protection. The roadway would be raised 4-ft above existing ground. The estimated cost would be $198,000.

9.2 San Felipe Creek Solutions

Previous efforts at flood control solutions in Del Rio have involved the buyouts of many homes in the San Felipe Creek watershed. In order to protect the natural beauty of San Felipe Creek, only certain solutions are being proposed. No major channel improvements or bypasses are expected to be feasible because of the environmental permitting obstacles. Also, the City is in the process of finalizing the San Felipe Creek Master Plan which is investigating ways to

enhance San Felipe Creek and develop economic benefits in much the same way as the San Antonio River Walk does for San Antonio. Given the fact that channel improvements are not feasible for San Felipe Creek, the remaining options for flood control are flow attenuation or bridge improvements. The largely undeveloped watershed north of the golf course provides several locations for regional stormwater facilities (RSWF).

9.2.1 Regional Stormwater Facilities

Regional Stormwater Facilities are super-sized detention ponds or could be described as small reservoirs. Depending on how the water is captured, or low flows are maintained, there are large variations in the permitting that would be required based on USACE regulations. There could be very little permitting required at all if the RSWF embankments do not actually encroach into the ordinary high water mark (OHWM), where the characteristic low flows are carried. A RSWF with this type of configuration would allow normal and low flows to pass through unaffected. It would only capture or impede the larger flood flows that spread out across the SFC floodplain. This type of flood control structure would remain dry during non-flood periods. As such, the normal flow regime hydrology would not be affected. At the other extreme would be a dam or retention structure, which would capture all of the flood flows and release very little flows if at all. The structure embankment would place a large amount of fill in the OHWM. This type of structure would have high environmental permitting impacts, such as an Individual Permit (IP). Individual Permits take a minimum of 18 months to complete, and cost hundreds of thousands of dollars to start out.

Site A

Site A is located in the lower portion of the watershed above the SFC Country Club, just upstream of Jap Lowe Drive. It would attenuate runoff from subareas SFC-05D, SFC-05C, SFC-05B and SFC-05A. The site controls runoff from 15.7 square miles of watershed, or 30% of the SFC watershed total. Analyses based on the most conservative assumption show that for complete capture of the flows the effective attenuation downstream would be a decrease of 9,000 cfs for the 100-yr ultimate storm event. Decreases of this magnitude correspond to WSEL decreases of approximately 2-ft in the near U.S. 90, with decreases in impact as you move downstream. **Table 9-2** illustrates the effects.

Table 9-2 – RWSF Site A - Summary of Peak Flow Reductions

Location	Ultimate Peak Flow (cfs)	Peak Flow with Detention (cfs)
CMB SFC15	40,234	31,460
CMB SFC20 (US 90)	40,193	31,707
Bottom	48,810	39,620

The opinion of construction cost is $3.1 million for this site, with more than half of that being associated with land acquisition ($1.7 million). A unit cost of $5,000 per acre was used for the cost estimate based on typical lots in the vicinity of Jap Lowe. The environmental permitting costs are expected to be quite high for complete capture of

runoff – there would be substantial impacts to wetlands and the OHWM. Also complete capture would require a large easement or purchase of land as the properties would be inundated for long periods of time. Complete capture would require the construction of an embankment structure 38-ft high. The construction of the Site A RSWF would require the purchase of 15 acres of land for the embankment, and an easement for hundreds of acres more in the inundation zone of the structure. A scaled down project with reduced flood mitigation would entail less environmental impacts, and also require less land acquisition. It is possible to reduce the permitting costs substantially, if the embankment is kept out of the OHWM and low flows are allowed to travel downstream undetained.

Site B

Site B is located at the next junction of several tributaries of San Felipe Creek downstream of Site A. The site controls runoff from 36.8 square miles of watershed, or 70% of the SFC watershed total. Analyses based on the most conservative assumption show that for complete capture of the flows the effective attenuation downstream would be a decrease of over 36,000 cfs for the 100-yr storm event, which is below the 2-yr event. The decreases are closer to the 10-yr event as you move to the bottom of the watershed. Decreases of this magnitude correspond to WSEL decreases of approx. 10-ft. Most of the bridge structures would no longer be inundated, except for Johnson Street and E. Academy Street. Table 9-3 illustrates the extreme effects of complete capture at Site B.

Table 9-3 - RWSF Site B - Summary of Peak Flow Reductions

Location	Ultimate Peak Flow (cfs)	Peak Flow with Detention (cfs)
CMB SFC20 (US 90)	40,193	3,577
Bottom	48,810	26,773

While Site B has an extreme impact on downstream peak flows, it also has a very large estimated cost of $18.5 million. The bulk of the cost of the project ($12.4 million) is associated with land acquisition. As with the Site A RSWF, the environmental permitting costs are expected to be quite high for complete capture of runoff – there would be substantial impacts to wetlands and the OHWM. Also complete capture would require a large easement or purchase of land as the properties would be inundated for long periods of time. Complete capture would require the construction of a dam structure nearly 70-ft high at the creek. The construction of the Site B RSWF would require the purchase of 50 acres of land for the embankment, and an easement for hundreds of acres more in the inundation zone of the structure.

9.2.2 Roadway Improvements

Johnston Street Bridge Replacement

The Johnston Street bridge is located immediately downstream of the Tardy Dam and associated park facilities. The low chord of the existing bridge is approximately 5-ft above the channel bottom. It is overtopped by 6-ft during the 2-yr flood.
To improve the safety of the structure, it is proposed to raise the road deck and approaches by approx. 9.5-ft, and construct a 150-ft long bridge. This will Provide a 5-yr level-of-service. The estimated cost would be $414,000. It would be difficult to provide much larger than 5-yr protection because of the shape of the floodplain. The 100-yr topwidth at this structure is about 1,300-ft wide and would require a substantially longer bridge to cross the floodplain.

E. Canal Street Bridge Replacement

The East Canal Street bridge has substantial bridge columns which are not oriented in the most efficient direction under the existing bridge. Demolishing the existing bridge and rebuilding the bridge with columns oriented parallel to the flow direction would substantially lower the energy losses through the bridge. This should also have the effect of decreasing channel velocities and scour potential. The low chord of the existing bridge is approximately 11-ft above the channel bottom. The existing bridge structure passes the 10-yr storm and is overtopped by more than 1-ft during the 25-yr flood.

There may be historical implications if the bridge is to be replaced.

E. Academy Street Bridge Replacement

The East Academy Street bridge is really more of a low water crossing than a bridge. It consists of four (4) 10'x4' MBC. The low chord of the existing bridge is approximately 1-ft above the normal pool of the creek. It is overtopped by almost 7-ft by the 2-yr flood event.
It is proposed to raise the road deck 7.5-ft and the approaches by approx. 5.0-ft, and replace the bridge class culverts with concrete slab bridge 150-ft long. This will Provide a 2-yr level-of-service. The estimated cost would be $355,000. The 100-yr topwidth at this structure is about 1,400-ft wide and would require a substantially longer bridge to cross the floodplain.

9.2.3 Buyouts

Buyouts have been employed in the past in the San Felipe Creek watershed, especially after the 1998 flood event. Buyouts are not being considered at this time.

9.3 San Felipe Creek Tributary 1

The San Felipe Creek Tributary 1 watershed generally has its headwaters west of Veterans Blvd and in the vicinity of Highland and East 17th Streets. There are no well-defined channels until the ditch along Airport Blvd. which drains to the south and ultimately joins several other ditches until they outfall to the mainstem of San Felipe Creek near the Port of Entry. Most of the runoff in the upper part of San Felipe Creek Tributary 1 flows via the streets through the neighborhoods because of the lack of existing stormsewer systems.

9.3.1 West 10th Street Stormsewer

There has been frequent flooding of the neighborhood just north of W. 10th Street because of the flat slopes. Additional culvert capacity has been added at the intersection of W. 10th and Airport Blvd., but water still backed up in Avenue U and Sage Drive.

The proposed improvements would add area inlets at Avenue Q and 13th Street to capture runoff as high in the neighborhood as possible. The collected runoff would be piped in over 3,100-ft of 3'x2' box culvert until it discharges into the existing channel south of W. 10th and Airport Blvd. The estimated costs for this solution are $1.14 million. As the box culvert is the most expensive part of the solution, tradeoffs between how much neighborhood is affected and the length of culvert can reduce the project cost. The capacity of the 3'x2' box culvert is approximately the 2-yr storm event.

9.4 Cienegas Creek Tributary 1

The Cienegas Creek Tributary 1 watershed generally has its headwaters south of W. Cantu Road and west of the high school. The main conveyance crosses under Wildcat Drive before bisecting the airport runways in a large culvert. Outside of the airport property, it leaves the City Limits and ultimately joins the mainstem of Cienegas Creek upstream of Vega Verde Road.

9.4.1 Wildcat Drive

The overtopping of Wildcat Drive by Cienegas Creek Tributary 1 occurs at the low water crossing near the high school baseball field. The existing two (2) undersized 36" CMP and the nearly at grade profile cause frequent overtopping. It is proposed to replace the existing CMPs with four (4) 72-inch RCPs and to raise the road profile by 2.0-ft. This combination of increased culvert capacity and raised road profile will provide 100-yr level of service at this crossing. The estimated construction cost is $182,000.

9.5 Cantu Branch Solutions

9.5.1 Regional Stormwater Facilities

Site D

Site D is located in the Cantu Branch portion of the Cienegas Creek watershed, just west of the neighborhood near Lausen Lane. The site would control runoff from 4.8 sq. miles out of 23.6 sq. miles in the watershed, or about 20% of the area contributing runoff downstream. Potential capture reductions are great at this site. Complete capture would reduce flows downstream in Cantu Branch by almost 5,000 cfs. Flow reductions of this magnitude correspond to water level decreases of 2-3 feet. It would effectively reduce the 100-yr ultimate flow to the 10-yr peak flow level.

Table 9-4 - RWSF Site D - Summary of Peak Flow Reductions

Location	Ultimate Peak Flow (cfs)	Peak Flow with Detention (cfs)
CMB CB30 (Veterans)	8,688	2,801
CMB CB55 (Amistad)	8,634	2,957
CMB CB65 (King's Way)	8,694	4,006

Due to the flat nature of the topography, it would require approximately 5,400 LF of embankment with a maximum height of 9-ft. The majority of the embankment would act as a barrier to flows which had previously flowed through the neighborhood along Danielle Drive. This RSWF would protect the neighborhood from upstream flooding from the Cantu Branch and also decrease flows downstream. The opinion of probable cost is around $2 million, with half of that cost being land acquisition for a 9-ft tall structure. As with the other RSWF sites, there will be a balance between stormwater capture and environmental permitting costs. Depending on the amount of attenuation designed for, the land requirements would vary. Complete capture (like a dam) would impact the most properties, while partial capture would decrease the inundation area but reduce the flood flow reduction.

9.5.2 Roadway Improvements

King's Way near Mary Lou

There are 3 project sites along King's Way for roadway improvements. One is on the Cantu Branch mainstem, and two others are on the Tributary 1 branch.
Site 1 involves replacing the low water crossing at Cantu Branch mainstem with 7 – 6'x5' MBC. This will provide 2-yr storm protection. The roadway would be raised 6-ft above existing ground. The estimated construction cost is $299,000.

Margaret Lane

The proposed project at Margaret Lane involves replacing the low water crossing at Cantu Branch mainstem with 5 – 7'x4' MBC. This will provide 2-yr storm protection. The roadway would be raised 5-ft above existing ground. The estimated construction cost is $120,000.

Amistad Blvd

Amistad Boulevard is inundated at each end. The western point of inundation is very near Kings Way. The proposed solution for Amistad near Kings Way is to replace the low water crossing with 6 – 8'x3' MBC. This would provide a 50-yr protection. The roadway would be raised 4-ft above existing ground. The estimated cost would be $230,000.

9.6 Cantu Branch Tributary 1 Solutions

9.6.1 Roadway Improvements

King's Way Sites 2 and 3

The other two sites along King's Way are located on Cantu Branch Tributary 1.
Site 2 involves replacing the low water crossing at CBT1 with 3 – 5'x3' MBC. This will provide 2-yr storm protection. The roadway would be raised 5-ft above existing ground. The estimated construction cost is $289,000.
Site 3 involves replacing the northernmost low water crossing at CBT1 with 2 – 7'x3' MBC. This will provide 10-yr storm protection. The roadway would be raised 5-ft above existing ground. The estimated construction cost is $194,000.

Amistad Blvd Site 2

Two possible solutions are proposed for Amistad Boulevard where it crosses Cantu Branch Trib 1 near King's Way. One proposed solution for Amistad near Kings Way is to replace the low water crossing with 6 – 8'x3' MBC. This would provide a 50-yr protection. The roadway would be raised 4-ft above existing ground. The estimated cost would be $226,000. If a bridge was employed, the cost would increase to $448,000.

King's Way /Amistad Blvd Intersection

Flooding from undetained runoff from the neighborhood adjacent to Echo Valley crosses the King's Way / Amistad intersection. Grate inlets across Echo Valley would capture runoff. Stormsewer lines consisting of 5'x5' SBC would carry the runoff along the property line between the nursing home and the church. The stormsewer line would outfall into Cantu Branch Tributary 1. The estimated cost would be $367,000.

Table 9-5: Regional Stormwater Facility CIPs - Summary of Benefits and Costs

CIP No.	Creek/ Tributary	Detention Site	Potent. % Reduction in Peak Flow	Description of Benefit	Opinion of Probable Cost
S5	**San Felipe Trib 1**	Site A	19	Flow Reductions; WSEL decr. 2-ft	$3,156,000
S6	**San Felipe**	Site B	45	Flow Reductions; WSEL decr. 9-ft	$18,514,000
C2	**Cantu Branch**	Site D	46	Flow Reductions; WSEL decr. 3-ft	$2,017,000
S4	**Calaveras Trib 1**	Vitela St	NA	Peak shaving; Water Quality	$345,000

Table 9-6: Roadway CIPs – Summary of Proposed Improvements

CIP No.	Creek/ Tributary	Street	Existing Structure Type/Size	Proposed Structure Type/Size	Proposed Change to Roadway Vertical Profile
S1	**Calavaras**	Railway Ave	1 - 48″ x 36″ CMP	2-6′ x 5′ MBC	Raise 4.5 ft
S2	**Calaveras Trib 1**	Plaza Ave	1-24″ CMP	2-6′x4′ MBC	None
S3		Bowie St	1-60″ CMP	2-8′x4′ MBC	None
S8	**San Felipe**	Johnson St	60′ Span Bridge	150′ Span Bridge	Raise 9.5 ft
S9		Canal St	270′ Span Bridge	260′ Span Bridge	Raise 2 ft
S10		E Academy St	4-10′x4′ MBC	150′ Span Bridge	Raise 5 ft
C1	**Cienegas Trib 1**	Wildcat Dr	2-36″ CMP	4-72″ RCP	Raise 2 ft
C3	**Cantu Branch**	King's Way	Low Water	7 – 6′ x 5′ RCB	Raise 6 ft
C5		Margaret	10 – 24″ CMP	5 – 7′ x 4′ RCB	Raise 5 ft
C6		Amistad Blvd	8 – 12″ CMP	6 – 8′ x 3′ RCB	Raise 4 ft
C7	**Cantu Branch Trib 1**	King's Way	Low water	3 – 5′ x 3′ RCB	Raise 5 ft
C8		King's Way	Low water	2 – 7′ x 3′ RCB	Raise 5 ft
C9a		Amistad Blvd	Low water	6 – 8′ x 3′ RCB	Raise 4 ft
C9b		Amistad Blvd	Low water	100 ft Span Bridge	Raise 4 ft
C10		King’s Way/ Amistad Blvd Intersection	Intersection	5′x5′ SBC Storm Sewer	N/A

Table 9-7: Roadway CIPs – Summary of Benefits and Cost Opinions

CIP No.	Creek/ Tributary	Street	Existing Level Of Service	Proposed Level Of Service	Opinion of Probable Cost
S1	**Calavaras**	Railway Ave	< 2-yr	2-yr	$198,000
S2	**Calaveras Trib 1**	Plaza Ave	<2-yr	2-yr	$144,000
S3		Bowie St	<2-yr	5-yr	$124,000
S8	**San Felipe**	Johnson St	<2-yr	2-yr	$413,000
S9		Canal St	10-yr	25-yr	$994,000
S10		E Academy St	<2-yr	2-yr	$355,000
C1	**Cienegas Trib 1**	Wildcat Dr	< 2-yr	10-yr	$182,000
C3	**Cantu Branch**	King's Way	< 2-yr	2-yr	$299,000
C5		Margaret Lane	< 2-yr	2-yr	$120,000
C6		Amistad Blvd	< 2-yr	50-yr	$230,000
C7	**Cantu Branch Trib 1**	King's Way	< 2-yr	2-yr	$289,000
C8		King's Way	< 2-yr	10-yr	$194,000
C9a		Amistad Blvd	< 2-yr	50-yr	$226,000
C9b		Amistad Blvd	< 2-yr	50-yr	$448,000
C10		King's Way/ Amistad Blvd Intersection	< 2-yr	N/A	$367,000

10.0 RECOMMENDATIONS

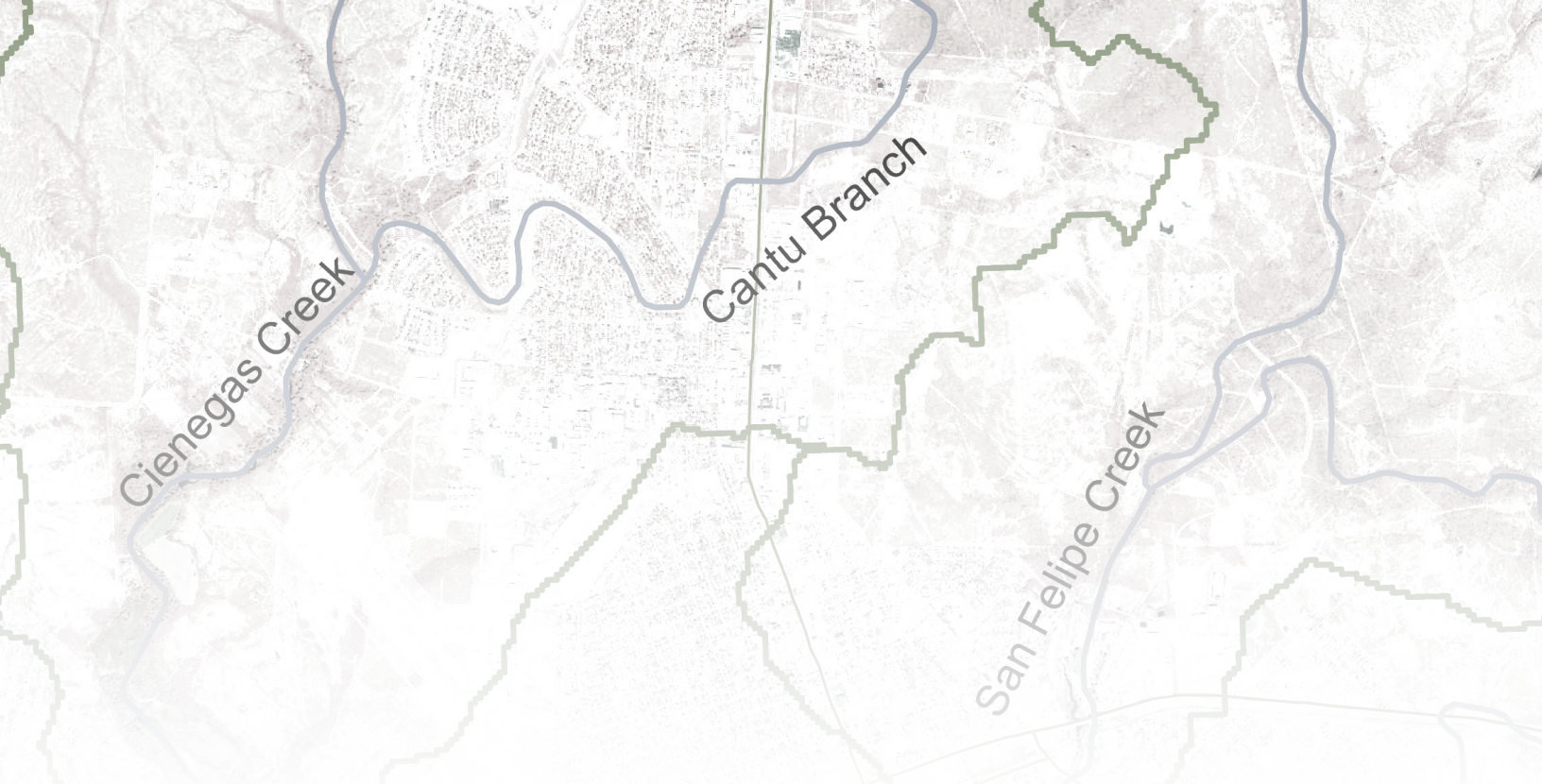
Based on the types of projects described in Chapter 9, there are two basic forms of projects that will have the most impact for public safety and ease of construction. The removal of roadways from the floodplain limits for the most frequent storm events will not only keep the most people out of harm's way but will also reduce the burden on the City staff who barricade problem areas and save labor costs. By providing 2-, 5- or 10-yr protection for many of the roads, traffic patterns can continue unabated except for the more extreme storm events.

The other type of project which would impact flooding issues for the future would be the construction of a regional stormwater facility in the open space in the upper portion of the San Felipe Creek or Cantu Branch watersheds. The ability to reduce flows can only occur where there is open space, and within the City Limits there are not that many available. With so little room in the City for mitigation of flood flows, the only options would be channelization or large box culverts. The channelization would incur large environmental permitting costs because of wetland or 404 issues, and the material costs to replace natural stream conveyance with underground culverts are considerable. A regional stormwater facility would reduce flood flows by delaying the volume of runoff, and allow it to drain at a slower more sustainable rate. The reduction of flows by a RSWF would also provide additional complementary benefits to the improved road crossings by increasing their level of service.

TABLE 10-1 RECOMMENDED PROJECTS

Project ID	Description	Est. Cost
Calaveras Creek		
Railroad Avenue	Road/Culv Imp.	$198,000
Plaza/W.Viesca St	Road/Culv Imp.	$144,000
Bowie Street	Road/Culv Imp.	$125,000
San Felipe Creek		
RSWF A	Regional Detention	$3,156,000
Johnson Street	Bridge Replacement	$414,000
Academy Street	Bridge Replacement	$354,000
Cienegas Creek Tributary 1		
Wildcat Drive	Road/Culv. Imp.	$182,000
Cantu Branch		
RSWF D	Regional Detention	$2,017,000
Kings Way– Site 1	Road/Culv Imp.	$299,000
Margaret Lane	Road/Culv Imp.	$120,000
Amistad Blvd	Road/Culv Imp.	$226,000
Cantu Branch Trib 1		
Kings Way– Site 2	Road/Culv Imp.	$289,000
Kings Way– Site 3	Road/Culv Imp.	$194,000
Amistad– Site 2b	Road/Culv Imp.	$226,000

The total of the 14 projects listed above is $7.9 million. Coordination with the City to prioritize the projects as part of a capital improvements program will likely change the order and composition of the list. Besides staff coordination, it is recommended that City Council be provided workshops to discuss the proposed projects and their benefits before real funding decisions are made. The City already has potential FEMA funding sources of $500,000 available to begin implementation of the roadway improvements. Further studies including traffic counts will be required as part of the benefit / cost analyses required by FEMA.



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