Appendix D Investigation and Analysis Report

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Dam Existing Condition and Concept Design Hydrology and Hydraulics Investigation and Analysis Report

Escondido Creek Floodwater Retarding Structure No. 1 Rehabilitation Project

Karnes County, Texas

U.S. Department of Agriculture - Natural Resources Conservation Service

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Delivering a better world

Prepared for:

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Table of Contents

Figures

Tables

Exhibits

1. Introduction

Escondido Creek FRS No. 1 is a Natural Resources Conservation Service (NRCS) designed dam built in 1954. It is located in southwest Karnes County within the Escondido Creek watershed. The purpose of this report is to capture the existing state of the dam and appurtenances, to document the best available data and methodology used in the Hydrologic and Hydraulic (H&H) analysis of the existing condition and the preferred alternative, and to present the results.

Escondido Creek FPS No. 1 requires upgrade based on the following concerns:

- The dam does not meet the current safety and performance criteria for high hazard dam.
- Downstream risk from a catastrophic breach. Approximately 1 house and 2 roads are within the sunny-day breach boundary.
- Extend the reservoir useful life by at least 50 years, preferably by 100 years.

2. Description of Existing Dam

The following sections reporting the existing conditions of FRS No. 1 is a compilation of the Dam Assessment Report (AECOM, 2014), Dam Safety Inspection Reports (SARA, 2021 and NRCS, 2023), and the FRS No. 1 As-built plans (USDA SCS, 1954) in addition to observations made during site visits associated with this Supplemental Watershed Plan (SWP) effort. All elevations referenced in this report are given in feet, North American Vertical Datum (NAVD 88), unless otherwise noted.

2.1 Current Condition of the Dam

FRS No. 1 is located in southwest Karnes County, Texas approximately 3 miles south and 3 miles west of Kenedy, Texas. The FRS is located on Panther Creek, a tributary of Escondido Creek, and a tributary of San Antonio River. The FRS is located in series approximately 3.5 miles upstream of FRS No. 13. FRS No. 1 is a typical NRCS earthen embankment dam with storage allocated for sediment accumulation and flood control. The 2021 Dam Safety Inspection Report (SARA, 2021) classifies the dam as Not Unsafe and Good condition with Adequate operation and maintenance. Similarly, the dam safety inspection report from 2022 (NRCS, 2023) classifies the dam as Not Unsafe and Very Good condition with Very Good operation and maintenance. Observations from the Dam Safety Inspection Report (NRCS, 2023) are included in **Section 2.5** through **Section 2.7**. The following items to monitor were noted in the inspection report.

- Watch for increased harvester ant and animal burrowing, rooting, and trailing on the dam embankment.
- Continue to control encroaching woody weeds, brush, and trees as they become evident on the dam embankment, the auxiliary spillway, the plunge basin, and around the principal spillway inlet and outlet.
- Watch for erosion where the grass is sparse or thin on the auxiliary spillway.

- Monitor the downstream slope over the principal spillway alignment for deterioration or movement of the early-stage slope slide.
- Monitor the joint near the top of the principal spillway inlet for leakage when the reservoir is filled.
- Continue to monitor the foundation drains.
- Monitor the pipe structure found at the upstream end of the pool left and right of the principal spillway inlet so that it does not cause an obstruction in the principal spillway.

The Sponsor is aware of the items noted above. These observations are not impacting the performance of the dam and are not the cause of the needed dam rehabilitation.

2.2 Potential Dam Safety Deficiencies

FRS No. 1 was constructed in 1954 to be a single-purpose, low hazard potential dam. Because there is a potential risk for loss of life downstream due to residential development and several roads should the dam breach, the structure is now classified as a high hazard potential dam. Currently the dam does not have the auxiliary spillway capacity to safely pass the Freeboard Hydrograph (FBH) for a high hazard potential dam without overtopping the embankment. In addition, the dam does not meet the 10-day drawdown requirement and engages the auxiliary spillway during the Principal Spillway Hydrograph (PSH) event.

2.3 Status of Operations and Maintenance

Dam operation and maintenance (O&M) of FRS No. 1 is performed by the San Antonio River Authority (the River Authority) via a contracted agreement between the Escondido Watershed District and the River Authority with 50% cost share. The last two dam safety inspections were completed by San Antonio River Authority on March 9, 2021 and by NRCS on February 22, 2022. The latest O&M inspection was conducted October 1, 2020.

Maintenance activities generally consist of repairing surface erosion, clearing brush from the auxiliary spillway and embankment slopes, mowing grass, and access road maintenance. Maintenance activities, such as mowing and tree removal, access road repair, and repair of animal burrows, are reported to be periodically performed. Based on the site visit on January 30, 2024, O&M on FRS No. 1 is considered adequate.

2.4 As-Built Dam Specifications

FRS No. 1 was designed and constructed in 1954 to be a single-purpose, low hazard potential dam. The embankment is a single zone, compacted earthfill dam and is noted to have an impervious core. A cutoff trench with 1:1 side slopes that has a minimum bottom width of 12 feet was constructed at the centerline of the dam. The dam is approximately 36 feet tall and 3,425 feet long. The upstream and downstream slopes of the embankment have a slope of approximately 3:1 and 2:1 (horizontal: vertical), respectively. The top width of the structure is approximately 14 feet. The land upstream of FRS No. 1 is predominantly private ownership.

Table 2.1. summarizes the as-built and existing structural data for FRS No. 1.

Table 2.1. As-Built and Existing Structural Data for FRS No. 1

Item	FRS No. 1	
	As-Built[a]	Existing[b]
Latitude / Longitude	28.7785° / -97.8954°	
Site Number	TX02032	
Year Completed	1954	
Purpose	Flood Control	
Drainage Area (mi^2)	2.84	3.05/ 3.22[c]
Dam Height (ft)	38.1	
Dam Type	Earthfill	
Dam Volume (yds^3)	97,954	
Dam Crest Length (ft)	2,310	2,310
Total Capacity (ac-ft)		
Sediment Submerged (ac-ft)	298	228[e]
Sediment Aerated (ac-ft)	0	0
Floodwater Retarding (ac-ft)	778	934
Surface Area (ac)		
Low Stage Principal Spillway (ac)[d]	46	32
High Stage Principal Spillway (ac)	52	41
Flood Pool (ac)[f]	112	110
Principal Spillway		
Type	Drop inlet, Two Stage	
Riser Height (ft)	10	
Conduit Size (in)	12	
Low Level Port Elevation (ft)	366.17	366.17
Riser Crest Elevation (ft)	368.17	368.17
Capacity at Aux Crest (cfs)	10.85	10.89
Energy Dissipater	Plunge Pool	Plunge Pool
Auxiliary Spillway		
Type	Earthen channel with protective vegetative cover	
Auxiliary Spillway Width (ft)	250	
Normal Pool (Low Stage) Elevation (ft)	366.17	366.17
Principal Spillway Crest Elevation (ft)	368.17	368.17
Flood Pool Elevation (ft)	377.87	378.02
Top of Dam Elevation (ft)	382.87	382.87
Datum.[a,b]	NAVD88	

a. As-built elevations are referenced to NGVD29 and were updated to NAVD88 datum Geoid 12B for this plan using conversion factor of +0.17 feet.
b. No site topographic survey was performed as part of this plan; all analysis was based upon Light Detection and Ranging (LiDAR) 2019 data.
c. The as-built drainage area is 2.84 mi^2 and the existing drainage area based on natural topography is approximately 3.05 mi^2. Factoring in the land contouring in the upper part of the basin which redirects more flow into the watershed, a more conservative area of 3.22 mi^2 was utilized for present day analyses.
d. The as-builts identify the sediment pool as the volume at the principal spillway crest. Aerated sediment storage above the principal spillway crest is not included in the as-builts.
e. The storage below 355.38 feet (the LiDAR water surface elevation at the time of data collection) are not included in the total capacity listed.
f. The flood pool is defined at the elevation of the auxiliary spillway crest.

2.5 Principal Spillway

The principal spillway inlet structure is a drop inlet (30 inches x 30 inches x 10 feet tall) with a steel debris guard and crest of 368.17 feet. There is one low-level port on the front side of the riser (16 inches tall x 24 inches wide) at elevation 366.17 feet. The conduit is 132-feet long of 12-inch-diameter of reinforced concrete pipe and 80-feet of 12-inch-diameter of corrugated steel metal pipe with three anti-seep collars.

The latest NRCS dam safety inspection report (2023) indicated that the visible section of the concrete principal spillway inlet and outlet was in good condition. The principal spillway conduit had good alignment near the outlet. An area of possible leakage through the joint near the top of the riser was noted. The inlet filter housing was functional with some rusting, also slight rust was noted on the slide gate. Johnsongrass around the inlet and small trees (1 inch diameter) on the right side of the outlet pipe were observed. A shallow hole upstream and to the left of the inlet was also located during inspection. The rock riprap around the outlet pipe was found to be in good condition with some burrows around the rock. Mesquites are growing downstream of the plunge basin, but the trees are not restricting flow at this time.

Photographs of the existing principal spillway system, taken during a site visit on January 30th, 2024, are provided in **Figure 2.1**. A low-level drain with a 16-inch by 24-inch opening resides on the upstream side of the riser. At the top of the riser, there is a debris guard and baffle wall cover. On the opposite side of the baffle wall, a valve stem extension is situated, which is linked to the gate valve at the bottom of the inlet riser.

Inlet structure looking southwest

Inlet structure looking northeast

Inlet structure and inlet filter housing

Plunge pool, currently dry

Figure 2.1 FRS No. 1 Principal Spillway Inlet and Outlet

2.6 Auxiliary Spillway

The auxiliary spillway is a 250-foot-wide, grass-lined channel with 3H:1V side slopes. The as-built drawings show the auxiliary spillway as having a flat grassed inlet section with 0% slope up to the control section, a 50-foot-long control section at elevation 377.87 feet (per as built) or 378.02 feet (per LiDAR), and an exit section with slopes ranging from 1.75% to 9.61% extending for approximately 1078 feet before transitioning back to the original ground. A channel was excavated at the end of spillway for relief wall drainage. Per correspondence with the San Antonio River Authority, there is no record of the auxiliary spillway flowing since dam construction.

According to NRCS dam safety inspection report (2023), all three sections of the auxiliary spillway were found to be in good dimensional condition with good vegetative cover. Some sparse areas were observed in the control section where the grass was slightly thin. An ATV trail was also noted across the control section. A harvester ant bed was found on the inside berm at the left end of the dam. The fence around the spillway was recorded as in good condition. Auxiliary spillway photos are provided in **Figure 2.2**.

Auxiliary spillway control section

Auxiliary spillway channel downstream of control section

Figure 2.2 FRS No. 1 Auxiliary Spillway Condition

2.7 Embankment

Per the NRCS dam safety inspection report (2023), the upstream and downstream embankments were observed to be in good condition with good vegetation covering most of the dam but with some slightly bare grass on the right end of the dam (looking downstream). Several deep burrows were noted on the dam up to 4.5 feet deep, along with a harvester ant bed measuring 2.5 feet diameter on the crown, 500 feet right of the principal spillway and 2.0 feet diameter at the upstream edge of crown, 150 feet left of relief well. Subterranean termites were also noted on the crown at the left end. A 3-feet by 3.5-feet hole was found ten feet down on the downstream slope, approximately 150 feet from the left end of the dam, with a smaller hole 5 feet to the right. Evidence of armadillo and old hog rooting were noted on the dam.

The crown is rounded at the downstream edge at the principal spillway alignment. An early-stage slope slide was visible over the principal spillway alignment on the downstream slope. A 6-foot-tall retama tree has sprouted mid-slope on the downstream embankment, roughly 200 feet to the right of the principal spillway alignment. An all-terrain vehicle (ATV) trail was located over the dam right of the monitoring station near the principal spillway. During the time of inspection, no livestock were present on the site and there was no water in the reservoir or the plunge pool. The downstream slope drain was clear with some minor sediment on the grate. Embankment photos are provided in **Figure 2.3**.

Upstream embankment

Downstream embankment

Downstream embankment showing burrows

Top of embankment

Figure 2.3 FRS No. 1 Embankment Condition

2.8 Topographic Data

No topographical survey was performed in support of plan development. A topographical survey may be performed as part of a future final design phase. Light detection and ranging (LiDAR) data combined with as-built elevations were the basis for critical elevations and the design of rehabilitative measures. The LiDAR data source that provided coverage for the analysis include:

- United States Geological Survey (USGS) Hurricane LiDAR 70-cm resolution LiDAR for Karnes County. Data was collected and processed by Fugro between January 4, 2019 through February 20, 2019 and published in June 2020.

The LiDAR coverage with respect to the location of FRS No. 1, the contributing watershed, and the area used for evaluation is shown in **Exhibit D-1.** The Mosaic tool in ArcGIS was used to combine the initial Digital Elevation Model (DEM) tiles into a single DEM at 1-meter resolution. The USGS Hurricane Lidar datasets were referenced to GEOID12B. The DEM was re-projected from UTM to Texas State Plane South Central coordinate system and elevations were converted from meters to feet. The re-projected DEM was used to verify as-built elevations (adjusted from NGVD29 to NAVD88) and to develop 1-foot interval contours for use in the analysis. The LiDAR DEM was also used to develop the elevation-storage relationship presented in **Section 2.9**. Bathymetric data were not collected for FRS No. 1 as part of the watershed planning effort.

2.9 Sediment and Reservoir Storage

FRS No. 1 was designed for a service life of 50 years with a sediment storage volume of 200 acre-feet below the low-level ports in the principal spillway riser**.** The single port set the normal pool surface area at 46 acres. The sediment storage was set at the elevation of the principal spillway crest, having 298.1 acre-feet of storage at elevation 368.17 feet (NAVD 88 adjusted). The surface area at the principal spillway riser crest was planned at 52 acres. The elevation-storage relationships from both as-built data and estimated from LiDAR (2020) data is provided in **Table 2.2**.

The low-level port in the 1954 original dam design was at 366.17 feet (NAVD88 adjusted), accounting for 200 acre-feet of sediment storage. A sediment survey has not been performed for this planning effort. The available LiDAR elevation allowed a conservative estimate of available sediment storage below the existing principal spillway crest for general dam performance knowledge and to aid in concept design.

At the time the LiDAR data was collected in early 2019, the WSE was at 355.38 feet. This elevation is 10.79 feet lower than the as-built low-level port elevation of 366.17 feet (NAVD88 adjusted). The assumption could be made that the water level is commensurate with the maximum amount of accumulated sediment deposited since the construction of the dam. According to the as-built elevation storage table, this equates to 86.43 acre-feet of accumulated sediment since construction in 1954 (64 years), assuming an assumed reservoir bottom of 347.17 feet per the dam centerline profile. The sediment accumulation rate is therefore approximately 1.35 acre-feet per year. Therefore, 100 years of future submerged sediment storage would be 135 acre-feet. To account for an additional 10 years between the 2019 LiDAR collection and the estimated rehabilitation construction completion (2019 to 2029), the total minimum submerged sediment storage volume needed is 149 acre-feet.

The available sediment storage at the principal spillway crest (per as-built table) at the time of the original dam construction is 298.05 acre-feet. The accumulated estimated maximum sediment of 86.43 acre-feet at the time the LiDAR was flown (i.e., below the water surface) plus the projected 149 acre-feet is only 235.43 acre-feet, less than the 298.05 acre-feet originally planned at the PS crest. Using the LiDAR storage above the WSE indicates that there is approximately 227.5 acre-feet available below the existing principal spillway crest elevation of 368.17 feet, which also exceeds the projected storage required of 149 acre-feet for future submerged sediment. Either estimate used (i.e., as-built storage or LiDAR storage above the WSE) indicates there is 100-years available future submerged sediment storage at Escondido FRS No. 1 below the elevation of the existing principal spillway crest.

Based on this evaluation, the principal spillway crest was maintained at 368.2 feet, rounding up 0.03 feet from the existing PS crest elevation of 368.17 feet for FRS No. 1, providing 228.8 acre-feet of available submerged sediment.

No aerated sediment volume was included above the principal spillway crest in the as-builts. For concept design modeling, aerated sediment storage volume above the principal spillway crest was included at 14% of the combined required sediment storage (149 acre-feet submerged plus 25 acre-feet aerated). These 25 acre-feet of aerated sediment sets the starting water surface elevation for the design runs at 368.78 feet.

Table 2.2. As-Built and Existing Storage for FRS No. 1

Notes	Elevation (ft NGVD 29)	Elevation (ft NAVD 88)	Storage As-Built (ac-ft)	Storage Current (ac-ft)
Assumed Bottom at Construction	347.00	347.17	0.00	0.00
WSE at time of LiDAR Collection	355.21	355.38	86.43	0.00
Permanent Pool/Low Level Elev	366.00	366.17	200.00	154.94
PS Crest / Sediment Pool	368.00	368.17	298.05	227.50
PS Crest Concept Design	368.03	368.20	299.69	228.81
Aerated Sediment Concept Design	368.61	368.78	330.9	253.80
	369.00	369.17	352.00	271.70
	373.00	373.17	626.00	509.32
	377.00	377.17	999.80	859.04
AS Crest	377.70	377.87	1076.24	933.95
AS Crest per LiDAR	377.85	378.02	1095.20	950.21
	381.00	381.17	1492.20	1344.95
Dam Crest Effective	382.70	382.87	1746.18	1598.35
	383.00	383.17	1791.00	1646.57
	384.00	384.17	1990.40	1814.32
	385.00	385.17	2189.80	1994.20

3. Hydrology and Hydraulics

A hydrology model for the Escondido Creek watershed was used to estimate flows for economic evaluation of the impacts of the considered alternatives. The recently completed Draft Karnes County Flood Protection Planning (FPP) (Doucet, 2023) was selected to use for this analysis, with edits as described below for the evaluation of Escondido Creek FRS No. 1 and the two concurrent Supplemental Watershed Planning efforts for FRS No. 4 and FRS No. 12 along Doe Branch and Bucker Creek, respectively. These models are prepared by the River Authority, a FEMA Cooperating Technical Partner (CTP).

3.1 Draft Karnes County Flood Protection Plan Background

The Draft Karnes County FPP model encompasses 55 subbasins for Escondido Creek and its tributaries, of which 19 subbasins were associated with the three study streams for the three dams: Panther Creek, Doe Branch, and Bucker Creek. The overall approach was to review and update all hydrologic inputs for the 19 subbasins associated with these three dams and three streams with the level of detail and methods commonly used for NRCS dam design. Outside of the detailed study area associated with Panther Creek, Doe Branch, and Bucker Creek, all subbasin parameters remain unchanged from the Draft Karnes County FPP, with the single exception of minor adjacent subbasins revisions for nine basins to match the revised watershed boundaries for this study. The Draft Karnes County model was an update to the model used to develop the 2007 DFIRM map (FEMA), which was also prepared by the River Authority using methods outlined in per the Draft San Antonio River Basin (SARB) Regional Modeling Standards for Hydrology and Hydraulic Modeling (the River Authority, 2018).

The final hydrologic models for Escondido Creek were not available at the time of this study. The River Authority concurred on the applicability of using these draft models for the economic evaluation of Escondido FRS No. 1. The draft models were reviewed by The River Authority and are close to the final version that will be used for the upcoming FEMA submittal.

There are two USGS gages on tributaries to the San Antonio River in the vicinity of Escondido Creek, 1) Gage 08186500 Ecleto Creek near Runge, TX, 2) Gage 08187500 Escondido Creek at Kenedy, TX. The Draft Karnes County FPP calibration focused on the Ecleto Creek gage, located approximately 8 miles northeast of the Escondido Creek gage, in close proximity to the study area. The Ecleto Creek flow gage has a long record spanning from 1903 to present day. The calibration effort used this record from 1903 to 2022 to perform a Bulletin 17B/17C model calibration. Additionally, storm event calibration was performed for four events in November 2002, March 2007, May 2015, and November 2018.

The calibration based on the Bulletin 17B analysis identified extreme streamflow events in 1903, 1952, 1967, and 1981 as "Historical" events, resulting in a 1% annual exceedance discharge of about 30,000 cfs, closely matching the HEC-HMS discharge of 29,000 cfs. The Bulletin 17C analysis, excluding pre-1962 records due to data gaps, initially indicated a 1% annual chance discharge of about 58,500 cfs using station skew, which is approximately double the HEC-HMS generated flow of 29,000 cfs. An alternative analysis based on TxDOT hydraulic Design Manual with a regional skew of 0.0 and MSE of 0.123 yielded a 1% annual chance discharge of approximately 42,000 cfs, which was within the 95% confidence interval. The Bulletin 17C analysis indicates that actual stream flow may be greater than the HEC-HMS predicted model flow.

For the storm event calibration, the Draft HEC-HMS model for Ecleto Creek overestimated the peak discharge for the November 2002 storm by approximately 41%, with the peak occurring 90 minutes earlier than observed, while for the March 2007 event, the peak discharge closely matched the observed value, although the peak time was later than observed. For the May 2015 and November 2018 events, the model underestimated peak discharges by 15% and 18%, respectively, with the peak times occurring more than 6 hours earlier and 90 minutes later than observed. On average, across all events, the modeled peak discharge was within 2% of the observed flow, and the time of peak was within 30 minutes of the observed peak (Doucet, 2023).

Based on the calibration results, the model parameters for the Ecleto Creek study area were considered representative of the entire study area. Since the same parameter development methods were applied across all watersheds, followed the procedures used in the development of the FEMA effective study, and aligned with the SARB regional modeling standards, no additional parameter adjustments were made by the River Authority. The Escondido Creek gage, recently installed in December 2015, does not have a sufficiently long record to perform a Bulletin 17B/17C analysis. Additional model calibration was not performed because additional watershed-wide parameter adjustments would not be reflective of the River Authority modeling approach for Karnes County and would only provide a minute amount of additional accuracy compared to the draft model.

3.2 Escondido FRS. No 1 Upstream Watershed

Section 3.2 discusses the parameter development for the area upstream of FRS No. 1. All other subbasins evaluated in detail along Panther Creek, Doe Branch, and Bucker Creek are discussed in **Section 3.3**.

Within the focused study area for Escondido FRS No. 1, 4, and 12 hydrologic parameters, including basin area, curve number, and lag time were updated in the Karnes County FPP hydrologic model for use in economics evaluation of this watershed plan. These changes were made following NRCS National Engineering Handbook guidelines to ensure consistent modeling parameters across both concept design analysis (SITES runs) and flooding impact analysis (HMS models). The subbasin area updates were minor and consistent with the Karnes County FPP (Doucet, 2023). The curve number re-estimation and the percent impervious cover closely aligned with the Karnes County FPP hydrologic model (majority of changes are less than 1.5 CN difference). The lag times were also reasonably consistent between models and are representative of the normal variation seen when using different lag time equations.

3.2.1 Subbasin Delineation

The FRS No. 1 upstream watershed was delineated based on 2019 LiDAR topography (USGS, 2020) and aerial imagery to ensure the inclusion of roadways and hydraulic crossing structures (e.g., culverts, bridges). The delineation on the south side of the subbasin required additional inspection and adjustment due to the presence of land contouring that redirects additional flow outside the natural drainage boundary back into the watershed that would normally be excluded (see **Exhibit D-2**). The contributing area based upon natural terrain was estimated to be 3.05 square miles. The contributing area considering this contouring was estimated to be 3.22 square miles using a combination of automatic delineation in GIS, engineering judgment, and hand edits.

3.2.2 Curve Number Loss Method

Curve numbers (CN) for Escondido Creek subbasins were estimated using the National Land Cover Database (NLCD) 2019 (Dewitz, 2021) and Soil Survey Geographic Database (SSURGO) (USDA, 2023) soil data per the guidance provided in the Draft San Antonio River Basin (SARB) Regional Modeling Standards for Hydrology and Hydraulic Modeling (the River Authority, 2018), Table 3.4, National Land Cover, Land Use Classifications and Corresponding TR-55 Classifications. The curve numbers for each NRCS TR-55 classifications were taken from the National Engineering Handbook (NEH), Part 630 Hydrology, Chapter 9, Hydrologic Soil-Cover Complexes. The resulting curve number correlation is provided per **Table 3.1**.

The NCLD 2019 land use layer (Dewitz, 2021) was manually adjusted in two ways. To ensure the roadways were accurately depicted, existing roadway extents were clipped to the NLCD 2019 land cover layer based on the available Karnes County parcel data (TxGIO, 2023). Second, the land use assignments upstream of FRS No. 1 were compared to the most recent imagery and minor adjustments were made to the assigned land use code. The land use map for the area upstream of FRS No. 1 is presented in **Exhibit D-6**. The hydrologic soil groups for the drainage area are comprised of predominantly Type B and C soils with minor inclusions of Type A and D soils (**Exhibit D-7**).

The percent impervious cover (%IC) was applied from San Antonio River Authority guidance provided on recent hydrologic studies (AECOM / Halff Associates, 2021 and 2022) where %IC is assigned to each NLCD code. The %IC assigned to each land use code is also provided in **Table 3.1**.

The area weighted curve number for the subbasin above Dam 1 is 70, the area weighted % IC is 3.71%, and the composite CN rounded to the nearest whole number is 71 (**Table 3.2**). For NRCS design, the average ARC curve number was adjusted per Figure 5A from the Engineering Technical Note No. 210-18-TX1 (USDA, 1982) to a value of 56.3, which is below the minimum CN allowed for NRCS design of 60. Therefore, an average ARC CN(II) of 60 was used for NRCS concept designs in setting the top of dam elevation.

Table 3.1. Escondido Supplemental Watershed Planning Curve Number, Impervious Cover, and Manning's n Assignments

NLCD No.	Classification	NEH Chapter 9 Classification	Hydrologic Condition	Curve Number by Soil Type				Impervious Cover %[c]	Assigned Manning's n	Notes
				A	B	C	D			
1[a]	Road	Road	-	98.0	98.0	98.0	98.0	100	0.050	Roads from county parcel data; assigned % IC from land use code 24
11	Open Water	Water	-	98.0	98.0	98.0	98.0	100	0.038	Draft SARB 2018
21[b]	Developed, Open Space	Open Space	Good	39.0	61.0	74.0	80.0	20[c]	0.040	SARB 2019 (NLCD class 21 appears to be class 85 in Draft SARB Regional Modeling Standards for Hydrology and Hydraulic Modeling (2018))
22[b]	Developed, Low Intensity	Open Space with COSA impervious percentage for 1/4 to 1 acre residential	Good	39.0	61.0	74.0	80.0	49[c]	0.090	Draft SARB 2018
23[b]	Developed, Medium Intensity	Open Space with COSA impervious percentage for 1/2 acre residential	Good	39.0	61.0	74.0	80.0	79[c]	0.120	Draft SARB 2018 does not have a developed medium intensity category so treated the same as low / high intensity with appropriate %IC.
24[b]	Developed, High Intensity	Open Space with COSA impervious percentage for 1/8 acre residential	Good	39.0	61.0	74.0	80.0	100[c]	0.160	Draft SARB 2018
31	Barren Land	Bare Soil / Newly Graded Areas	-	77.0	86.0	91.0	94.0	0	0.025	Draft SARB 2018
41	Deciduous Forest	Woods	Fair	36.0	60.0	73.0	79.0	0	0.150	Draft SARB 2018
42	Evergreen Forest	Woods	Fair	36.0	60.0	73.0	79.0	0	0.120	Draft SARB 2018
43	Mixed Forest	Woods	Fair	36.0	60.0	73.0	79.0	0	0.140	Draft SARB 2018
52[b]	Shrub/Scrub	Brush (Brush-Forbs-Grass Mixture) / Brush Major Element	Fair	35.0	56.0	70.0	77.0	0	0.038	Draft SARB 2018
71	Herbaceous/Grassland	Meadow	Good	30.0	58.0	71.0	78.0	0	0.038	Draft SARB 2018
81	Pasture/Hay	Pasture (Pasture, Grassland, or Range)	Fair	49.0	69.0	79.0	84.0	0	0.038	Draft SARB 2018
82	Cultivated Crops	Row crops (SR+CR)	Good	64.0	75.0	82.0	85.0	0	0.035	Draft SARB 2018
90[b]	Woody Wetlands	Woods	Good	30.0	55.0	70.0	77.0	100	0.098	Draft SARB 2018
95[b]	Emergent Herbaceous Wetlands	Meadows	Good	30.0	58.0	71.0	78.0	100	0.068	Draft SARB 2018

a. Roads w/right-of-way were overlain on the NLCD to ensure all roadways were captured and assigned a new land use code not listed in the SARB Regional Modeling Standards for Hydrology and Hydraulic Modeling (2018).

b. These NLCD categories have adjusted numbering since SARB Regional Modeling Standards for Hydrology and Hydraulic Modeling (2018) publication. The category numbers used align with the NLCD 2019 (Dewitz, 2021) codes and the classifications applied align with the SARB Regional Modeling Standards for Hydrology and Hydraulic Modeling (2018).

c. Impervious cover percentages were based upon the higher of the City of San Antonio (CoSA) average %IC per SARB Regional Modeling Standards for Hydrology and Hydraulic Modeling (2018) Table 3.2 or the more recent impervious cover guidance provided by the River Authority for 2019-2021 hydrologic modeling studies (AECOM / Halff Associates, 2021 and 2022) performed by AECOM. For NLCD codes 21 through 24, the %IC from the USAR and Medina studies was more conservative and used in this analysis.

3.2.3 Time of Concentration

The time of concentration (T_c) of the FRS No. 1 upstream watershed was estimated using both the NRCS Velocity Method and the NRCS Watershed Lag Method as described in NEH Part 630 Chapter 15 (NRCS 2010). The Velocity Method consists of the longest flow path broken up into sheet flow, shallow concentrated flow, and open channel flow segments. Each segment requires the length and slope as well as the land cover. Open channel geometry is needed for the channel segments as well. The Watershed Lag Method uses an empirical equation which requires the basin's watercourse length, average basin slope, CN, and subbasin area.

The Tc value of 1.39 hours used in the hydrologic analysis is based on the Velocity Method. For comparison, the Tc value estimated by the Watershed Lag Method is 2.25 hours. The longest flow path used in this analysis is shown in **Exhibit D-3**.

A summary of the hydrologic inputs for Escondido FRS No. 1 is presented in **Table 3.2.** The parameters used in previous hydrologic studies is also provided for comparison purposes only.

Table 3.2. Hydrologic Inputs for FRS No. 1

Parameter	AECOM (2024)	Dam Assessment (AECOM, 2014)	Draft Karnes County FPP (Doucet, 2023)
Drainage Area (sq. mi.)	3.22	3.17	3.02
Curve Number (Type II)	71	73.4	69.3
Curve Number (Type II Adjusted)	60[1]	58.8	NA
Time of concentration (hrs)	1.39	1.36	1.98

1/ Average Antecedent Runoff Condition (ARC) CN was estimated at 56.3 but rounded up to minimum of 60.
NA = Not applicable

3.2.4 Precipitation

Point rainfall for the frequency storm analysis and the probable maximum flood (PMF) analysis was obtained from the following three sources:

- National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Depth-Duration Frequency (DDF) Rainfall Values for Precipitation Area (PA) 8 from SARB Modeling Standards were used in frequency analysis in HEC-HMS. The PA-8 rainfall is used to be consistent with SARB Modeling Standards and analysis of other two other dams FRS No. 4 and FRS No. 12. Rainfall values are summarized in **Table 3.3**.
- NOAA Atlas 14, Volume 11, Version 2 (2018) rainfall depths over the subbasin were used for the FRS No. 1 concept design in SITES due to slightly higher rainfall depths compared to the SARB PA-8 rainfall depths (discussed below in **Section 3.4.1**). The NOAA Atlas 14 rainfall depth was also used for the 200-year storm event in frequency analysis, as SARB PA - 8 lacked data for this storm event. Rainfall values are summarized in **Table 3.4** and **Table 3.11**.
- TCEQ Probable Maximum Precipitation (PMP) rainfall depths were estimated with the Applied Weather Associates, LLC (AWA) web application Texas Basin PMP Tool (TCEQ, 2023). A summary of the PMP rainfall values for both the area contributing to FRS No. 1 and the combined area contributing to downstream FRS No. 13 is presented in **Table 3.5**.

Table 3.3. Escondido FRS No. 1 SARB NOAA Atlas 14 PA-8 Rainfall Values for Frequency Storm Modeling

Storm	Rainfall Depth (inches) for AEP Events						
Duration	50%	20%	10%	4%	2%	1%	0.2%
5 minute	0.53	0.65	0.76	0.90	1.02	1.13	1.37
10 minute	0.85	1.04	1.21	1.44	1.63	1.81	2.17
15 minute	1.07	1.31	1.51	1.79	2.02	2.24	2.71
30 minute	1.50	1.83	2.11	2.49	2.80	3.10	3.77
1 hour	1.96	2.40	2.78	3.31	3.73	4.15	5.13
2 hour	2.39	3.09	3.55	4.31	4.92	5.56	7.18
3 hour	2.65	3.51	4.03	4.97	5.72	6.53	8.67
6 hour	3.07	4.01	4.85	6.08	7.10	8.23	11.34
12 hour	3.48	4.57	5.58	7.11	8.40	9.87	14.10
24 hour	3.92	5.16	6.37	8.22	9.78	11.60	16.93

Table 3.4. Escondido FRS No. 1 NOAA Atlas 14 Rainfall Values for Frequency Storm Modeling and NRCS Design

Storm Duration	0.5% AEP Rainfall Depth (inches)
5 minute	1.25
10 minute	1.99
15 minute	2.48
30 minute	3.44
1 hour	4.63
2 hour	6.31
3 hour	7.50
6 hour	9.59
12 hour	11.60
24 hour	13.80

Table 3.5. Escondido FRS No. 1 TCEQ PMP Rainfall Values

Storm Duration (hr)	Above Dam 1 Rainfall Depth (inches)	Combined Area Above Dam 13 Rainfall Depth (inches)
1	11.60	11.50
2	21.60	18.60
3	23.50	21.40
6	29.50	27.40
12	35.90	35.40
24	43.10	42.50
48	46.20	45.70
72	46.20	45.80

3.3 Downstream Study Area

3.3.1 Project Setting and Data Sources

FRS No. 1 is located on Panther Creek, contributing to Escondido Creek, a tributary of the San Antonio River. The Draft Karnes County FPP hydrologic model in Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) Version 4.11 for Panther Creek, Doe Branch, and Bucker Creek tributaries was updated with revised drainage areas and hydrologic parameters.

The HEC River Analysis System (HEC-RAS) 1D hydraulic models for Panther Creek and Escondido Creek were also revised from the Draft Karnes County FPP hydraulic models for this analysis. The Panther Creek HEC-RAS model was trimmed at the toe of FRS No. 1, utilizing only the downstream segment for this analysis. The Escondido Creek HEC-RAS model was used with minor modifications, which extends to the confluence with the San Antonio River, covering a distance of 26.2 miles of Escondido Creek. Updates made to the hydraulic models is further discussed in **Section 3.3.3**.

The following data sources were used in the development of these models:

- TNRIS, USGS Hurricane LiDAR. Published June 2020
- National Land Cover Dataset (NLCD) 2019 (Dewitz, 2021)
- SSURGO Soils
- Field verification of second box culvert (northeast side) associated with the Panther Creek HEC-RAS modeling collected December 2023
- TxDOT Bridge Inventory Data for box culvert with four barrels on Panther Creek Crossing along SH 72
- As-built plans for Escondido Creek FRS No. 1.

3.3.2 Hydrologic Analysis

3.3.2.1 Subbasin Delineation

FRS No. 1 and FRS No. 2 are upstream and in series with FRS No. 13. The other six subbasins along Panther Creek to the confluence with Escondido Creek were updated from the Draft Karnes County FPP model (Doucet, 2023). The study area downstream of FRS No. 1 along Panther Creek and Escondido Creek was modeled in HEC-HMS Version 4.11. Like the subbasin above FRS No. 1, the subbasin boundaries on Panther Creek were revised using the 2019 LiDAR and checked against current aerial imagery to ensure that the presence of roadways and hydraulic crossing structures (e.g., culverts, bridges) were captured properly. All subbasins for Doe Branch and Bucker Creek for evaluation of FRS No. 4 and FRS No. 12, respectively, were also reviewed and updated.

Nine subbasins adjacent to the detailed study area were adjusted to align with the revised watershed boundaries used for this study. The area adjustments for adjacent Karnes County FPP (Doucet, 2023) subbasins are provided in **Table 3.6**. A map of the delineated Panther Creek subbasins is presented in **Exhibit D-3.** The subbasin areas for Panther Creek, Doe Branch, and Bucker Creek subbasins are provided in **Table 3.7** and shown in **Exhibit D-4**. **Exhibit D-5** shows the Escondido Creek Watershed Planning detailed study area along with the Draft Karnes County FPP (Doucet, 2023) subbasins.

Table 3.6. Drainage Areas for Adjacent Karnes County FPP Subbasins

Karnes County FPP Subbasin ID	Original Area (sq. mi.)	Revised Area (sq. mi.)
3040110	3.847	3.837
3040112	3.114	3.119
3040113	1.952	1.960
3040116	0.117	0.117
3040201	3.830	3.789
3040205	0.712	0.702
3040215	0.979	0.977
3040218	3.533	3.531
3040221	0.613	0.611

3.3.2.2 Escondido Dam Rating Curves

The Escondido HEC-HMS model includes 13 NRCS dams, including FRS No. 1. Nine of these dams are situated within the portion of the Escondido Creek watershed under evaluation for the three concurrent Supplemental Watershed Planning studies: FRS Nos. 1, 2, 3, 4, 5, 6, 7, 12, and 13. FRS Nos. 2 and 13 are positioned within Panther Creek, FRS Nos. 3 and 4 are located along Doe Branch, and FRS Nos. 5, 6, 7, and 12 are situated within Bucker Creek.

The structural parameters for the six dams not under detailed evaluation as part of the Supplemental Watershed Planning effort were established utilizing available as-built plans and the most recent topographic data for the auxiliary spillway rating curve profile (FRS No. 2, 3, 5, 6, 7, and 13). No modifications to the HEC-HMS rating curves in the Karnes County FPP model were made to the remaining four dams outside the detailed study area (FRS No. 8, 9, 10, and 11). The elevation-storage relationship for the nine dams in the detailed study area were estimated based on the topographic data described in **Section 2.8**. The elevation-storage and storage-discharge rating curves were added to the HEC-HMS frequency storm analysis model.

3.3.2.3 Curve Number Loss Method

Curve numbers within the Escondido Supplemental Watershed Planning study area were reevaluated using the CN correlation described in **Section 3.2.2**. The land use map is presented in **Exhibit D-6**. Similar to the FRS No. 1 subbasin, the hydrologic soil groups for the Escondido study area are comprised of predominantly Type B and Type C soils with minor inclusions of Type D and Type A soils.

The CNs for all updated watersheds along Panther Creek, Doe Branch, and Bucker Creek are summarized in **Table 3.7**. In general, the CNs and %IC are similar or slightly higher than the Karnes County FPP, indicating good agreement between the two studies.

3.3.2.4 Time of Concentration

The time of concentrations (Tc) for the remaining six subbasins on Panther Creek were estimated using the NRCS Velocity Method and the NRCS Watershed Lag Method as described in **Section 3.2.3**. A summary of the Velocity Method time of concentration analysis for the Supplemental Watershed Planning effort including Panther Creek (FRS No. 1), Doe Branch (FRS No. 4), and Bucker Creek (FRS No. 12) is presented in **Table 3.7**. The longest flow paths for Panther Creek are shown in **Exhibit D-3**.

3.3.2.5 Routing Reaches

The routing reaches downstream of FRS No. 1 along Panther, Doe Branch, and Bucker Creek were revised from the Draft Karnes County FPP. No changes were made to the HEC-HMS routing reaches outside the detailed study area in the Karnes County FPP model.

The Karnes County FPP model utilizes two reach routing methods: Modified Puls where HEC-RAS hydraulic models are available (i.e., Panther Creek and Escondido Creek) and Muskingum-Cunge for all other streams. The Muskingum-Cunge routing method is applied to reaches along Doe Branch, Bucker Creek, and the segment between FRS No. 2 and the junction with Panther Creek. For these reaches, the 2019 LiDAR topography provided data for a representative 8-point cross-section, reach slope, and reach length. Aerial photography was used to determine Manning's roughness values for the left and right overbanks. These inputs were then used for routing in the HEC-HMS model, and the resulting routing parameters are presented in **Table 3.8**.

The Modified Puls routing method is utilized for four reaches along Panther Creek. For this method, 22 increasing discharges were used to estimate the storage-discharges relationship ranging from zero to a flow greater than the estimated 500-year discharge in each hydraulic model. The estimation of storage-discharge relationships was carried out using a customized spreadsheet, which uses the HEC-RAS results as input. The HEC-RAS results were also used to estimate the flow velocity in the main channel. The number of sub-reaches within each reach was estimated by assuming that the floodwave velocity is 1.5 multiplied by the channel velocity and by using a time step of 5 minutes.

3.3.2.6 Precipitation and Areal Reduction

Precipitation depths from the combination of SARB Modeling Standards Precipitation Area (PA) - 8 and NOAA Atlas 14, Point Precipitation Frequency Estimates were used for the frequency storm analysis in hydrologic model as described in **Section 3.2.4**. The PA - 8 rainfall depths for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% Annual Exceedance Probability (AEP) storm events were utilized. NOAA Atlas 14 rainfall depths were used for the 0.5% AEP storm event only to meet the requirement of eight storm events in HEC Flood Damage Reduction Analysis (HEC-FDA) for economic analysis. The frequency storm events were compiled using a five-minute minimum storm intensity duration with peak intensity positioned at the center of the hyetograph (50%).

To account for areal reduction when the combined basin area exceeds 10 square miles in the HEC-HMS analysis, TP-40 area reduction was applied using the Depth-Area analysis option for key flow locations in the HEC-HMS model with greater than 10 contributing square miles. 36 depth-area analysis points, including subbasins, dams, and junctions, were selected for the depth area analysis for the eight AEP events listed above.

Table 3.7. Summary of Hydrologic Inputs for Panther Creek, Doe Branch, and Bucker Creek Subbasins

Name	Description	Area (sq. mi.)	Curve Number Loss Method			Transform Method		
			CN (II)	% IC	Composite CN (II) w/IC	Longest Flow Path Length (feet)	Time of Concentration (hrs)	Lag Time (min)
Bucker Creek								
BCK-001	Above Dam 5	1.341	82.12	8.18	84	8559	0.62	22.32
BCK-002	Below Dam 5	0.053	82.37	21.08	86	2898	0.44	15.84
BCK-003	Above Dam 6	2.414	81.74	8.97	83	15129	1.47	52.92
BCK-004	Below Dam 6	0.147	82.68	16.31	85	5192	0.72	25.92
BCK-005	Above Dam 7	1.820	78.97	10.36	81	9713	1.62	58.32
BCK-006	Below Dam 7	0.155	74.41	2.36	75	4058	0.73	26.28
BCK-007	Above Dam 12	5.709	76.29	10.41	79	26961	3.14	113.04
BCK-008	Below Dam 12	0.945	72.40	8.01	75	15637	1.47	52.92
Doe Branch								
DOE-001	Above Dam 3	4.559	75.84	5.25	77	19386	1.96	70.56
DOE-002	Above Dam 4	6.301	73.13	7.49	75	30587	2.95	106.2
DOE-003	Below Dam 4	0.886	74.28	19.49	79	15071	2.07	74.52
Panther Creek								
PAN-001	Above Dam 1	3.216	70.03	3.71	71	14664	1.39	50.04
PAN-002	Below Dam 1	0.269	68.98	15.39	74	4902	0.45	16.20
PAN-003	Above Dam 2	2.404	68.87	7.29	71	13470	1.74	62.64
PAN-004	Below Dam 2	0.346	69.97	2.90	71	6860	1.06	38.16
PAN-005	Above Dam 13	4.591	73.75	6.71	76	18343	1.51	54.36
PAN-006	Below Dam 13	0.205	69.80	4.23	71	3078	0.66	23.76
PAN-007	Below Dam 13	0.087	66.86	21.73	74	4641	0.51	18.36

Table 3.8. Routing Reach Parameters for Panther Creek, Doe Branch, and Bucker Creek

Reach	Length (ft)	Slope	Manning's n			Index Celerity (ft/s)
			Channel	Left Overbank	Right Overbank	
Panther Creek						
R_ PAN-004	5371	0.0048	0.040	0.100	0.100	5
Doe Branch						
R_ DOE-002	22938	0.0030	0.040	0.035	0.035	5
R_ DOE-003	13216	0.0028	0.040	0.050	0.070	5
Bucker Creek						
R_ BCK-002	2304	0.0063	0.040	0.035	0.035	5
R_ BCK-004	3423	0.0037	0.040	0.035	0.035	5
R_ BCK-006	3247	0.0054	0.040	0.035	0.035	5
R_ BCK-007	15915	0.0025	0.040	0.035	0.035	5
R_ BCK-008	11248	0.0020	0.040	0.050	0.050	5

3.3.3 Hydraulic Analysis

Analysis for the Escondido Supplemental Watershed Planning studies used both 1D steady flow and 2D unsteady flow hydraulic models. Both 1D and 2D hydraulic models were developed in HEC-RAS version 6.3.0. The 1D steady flow hydraulic model is used for frequency analysis during the eight designated storm events while the 2D unsteady flow hydraulic model was used for NRCS sunny day dam breach analysis and inundation mapping.

3.3.3.1 HEC-RAS 1D Frequency Storm Analysis

For FRS No. 1, HEC-RAS models obtained from the Draft Karnes County FPP (Doucet, 2023) study was used as the starting point for the Panther Creek and main stem Escondido Creek hydraulic models. The original Panther Creek HEC-RAS model was truncated at the FRS No. 1. The stream centerline immediately below the dam was realigned to follow the principal spillway outflow channel downstream of the dam. Cross sections for both Panther Creek and Escondido Creek HEC-RAS models were extended to contain the higher discharge exhibited with the federal decommission alternative. Cross section locations for Panther Creek and Escondido Creek are shown on **Exhibit D-8**.

All considered alternatives for detailed economic analysis (Alternative 1 - No Action, Alternative 2 - Decommission, Alternative 3 – High Hazard Potential Rehabilitation with Auxiliary Spillway Raise, and Alternative 4 – High Hazard Potential Rehabilitation with RCC Stepped Spillway) were simulated in the 1D HEC-RAS models for the eight frequency storm events discussed in **Section 3.3.2.6**. Flow data from frequency storm analysis were utilized for subbasins with a drainage area less than or equal to 10 square miles, while results from the depth-area analysis runs were used for subbasins and junctions with an aggregate drainage area greater than 10 square miles.

For Panther Creek, flow change locations were assigned via Excel spreadsheet using standard hydraulic modeling protocols, including flow changes at cross sections before roadway crossings, immediately downstream of dams, and one-third of the distance up a subbasin reach length from the outlet. For Escondido Creek, the flow change Excel spreadsheet from the Karnes County FPP effort was used to assign flows at the generally the same locations with minor updates.

The crossings along Panter Creek include those listed below:

- RS 24827 (Crossing SH 72) – field verified second box culvert (northeast side) due to realigned Panther Creek below FRS No. 1; no changes from the Draft Karnes County FPP
- RS 11678 (Crossing CR 163) – no changes from Draft Karnes County FPP
- RS 4571 (Crossing FM 2102) – no changes from Draft Karnes County FPP

The crossings along Escondido Creek were taken from the Draft Karnes County FPP. The crossings were reviewed, and the following crossings had Geoid adjustments from presumed Geoid 03 to Geoid 12B applied to the structure elevations:

- RS 117553 (Crossing CR 209)
- RS 112920 (Crossing CR 185)
- RS 101682 (Crossing CR 186)
- RS 95973 (Crossing FM 99)
- RS 84451 (Crossing FM 135)
- RS 53933 (Crossing US 181, Kenedy TX)
- RS 52239 (Crossing North 5th St, Kenedy TX)
- RS 47444 (Crossing FM 792, Helena RD, Kenedy, TX)

No changes from the Draft Karnes County FPP were made to the following crossings:

- RS 132920 (Crossing CR 210)
- RS 16296 (Crossing CR 331)
- RS 9606 (Crossing Private Road)

The downstream boundary condition for the Panther 1D HEC-RAS model was based on an estimated normal depth slope for the channel centerline, measured from the downstream cross section an equal distance both upstream and downstream. The Panther Creek boundary condition was estimated at 0.001 ft/ft. The boundary condition for Escondido Creek was unchanged from the Draft Karnes County FPP (Doucet, 2023).

3.3.3.2 HEC-RAS 2D Sunny-Day Breach Analysis

Technical Release No. 210-60 (TR-210-60) Earth Dams and Reservoirs (USDA NRCS, 2005) and TR-66 Simplified Dam-Breach Routing Procedure (NRCS SCS, 1985) breach criteria and procedures were used to estimate a breach discharge hydrograph. Fair weather conditions were assumed for the sunny day breach analyses, resulting in a peak discharge of 28,600 cfs. The initial reservoir pool elevation assumed for the breach scenario was static at top of dam with non-storm conditions downstream. The HEC-RAS 2D model was used to map the breach extent downstream of FRS No. 1.

The 2D HEC-RAS model for FRS No. 1 includes approximately 9.79 square miles of 2D flow area. Several breaklines were added along the crown of major roads and other elevated features such as existing dams and elevated channel banks to better define the terrain. Additionally, six SA/2D connections were incorporated to represent culverts and bridges along Panther Creek and Escondido Creek. Three of the six crossings were situated on Panther Creek, while the remaining crossings were located on Escondido Creek. All crossings utilized in the 2D HEC-RAS model were aligned with structures used in the 1D HEC-RAS. The geometry of the structures, including bridge openings, pier dimensions, culvert sizes, and lengths from the 1D HEC-RAS, was applied to the 2D flow area with the help of the SA/2D connection data editor. The terrain was created from the LiDAR dataset and aerial imagery. Manning's values were assigned based upon land used codes (as discussed in **Section 3.2.2**) per **Table 3.1**.

The inflow hydrographs for 2D analysis were applied as the upstream boundary condition for all three breach scenarios evaluated (i.e., static, hydrologic, and seismic) at the downstream toe of the dam. For the FRS No. 1 HEC-RAS 2D model, the downstream normal depth was estimated to be 0.003 ft/ft downstream of Kenedy, Texas.

The work areas use a base cell size of 100-foot, with 50-foot cells along prominent breakline features to define roads, railroads, embankments, and other high-ground features within the inundation boundary. Where necessary, refinement regions were added using a 40-foot cell size.

The hydraulic model was run using the full momentum, Shallow Water Equation – Eulerian-Lagrangian Method (SWE-ELM) equation set with a fixed time step of 5 seconds for a 24-hour simulation window. The model runs with a 1-minute mapping output interval, a hydrograph output interval of 1-minute, and a 1-minute detailed output interval.

Using the results of the sunny-day breach analysis, the population at risk (PAR) was estimated for the existing condition (i.e., with existing dam in place). Note that estimating the PAR is based on professional judgment coupled with empirical data. PAR estimates were provided for motorists, residents, and other people located downstream that could be affected by flooding from a catastrophic failure of FRS dam.

Guidance for Completion of "Evaluation of Potential Rehabilitation Projects" December 10, 2001, Updated January 2021 was utilized to estimate PAR for residences and motorists downstream of the dam. According to the guidance, three people per residence are estimated to be at risk where floodwaters are greater or equal to 1.0 foot above natural ground elevation. For paved roads with predominantly local traffic, one vehicle per road with two people per vehicle are estimated to be at risk where floodwaters overtop the road deck at a depth of greater than or equal to 1.0 foot.

The PAR for FRS No. 1 during a sunny-day breach was estimated to be 6. All rehabilitation options considered would eliminate or greatly reduce the risk to the population downstream to an acceptable level.

3.3.3.3 HEC-RAS 2D FBH Storm Event Breach Analysis

The FBH storm event breach analysis for Escondido FRS No. 1 was performed using HEC-HMS version 4.9 and downstream breach impact was analyzed in HEC-RAS 2D version 6.3. The hydrologic inputs included the existing dam rating curves for FRSs No. 1, 2, and 13, lag time, routing reaches, and adjusted Type II CN for all subbasins and their contributing drainage areas. The NRCS 6-hour FBH with standard NRCS rainfall distribution and NRCS 24-hour FBH with a 5-point rainfall distribution were used for the 6-hour and 24-hour FBH storm events, respectively.

HEC-HMS Breach Hydrograph Development

The hydrologic analysis indicated that both the 6-hour and 24-hour FBH storm events would overtop and breach FRS No. 1. However, the 24-hour FBH was the more critical storm event for FRS No. 13, so this storm event was used in evaluation of the dual breach of FRS No. 1 and FRS No. 13. Note that FRS No. 2, the dam parallel to FRS No. 1 and upstream of FRS No. 13, does not overtop during either the 6-hour or 24-hour FBH storm events and was therefore not breached for this analysis. The FBH breach flow from FRS No. 1 plus the FBH outflow from FRS No. 2, combined with the 24-hour FBH storm in the uncontrolled drainage area above FRS No. 13 causes Dam 13 to overtop its effective dam crest and breach.

To model the storm event catastrophic breach scenario, dam break analysis was conducted in FRS No. 1 and FRS No. 13 in HMS as shown in Figure 3.1. The breach configurations for FRS No. 1 and 13 were determined in accordance with Section 8.3 Breach Parameters of the TCEQ H&H Guidelines for Dams in Texas (2007) and are described below:

- The maximum width of the breach was considered three times the depth of the water impounded when the reservoir water level is at the top of dam.
- Side slopes of the breach were modeled as vertical.
- The breach time of failure was considered to propagate at a rate of three feet of depth of water impounded per minute.
- A overtop breach method was used as the dam failure method.
- Trigger elevation was set at top of dam elevation.

A summary of the modeled dam breach parameters and peak breach discharges are provided in **Table 3.9**. **Figure 3.2** shows a graph of the resulting NRCS FBH storm event breach hydrographs.

Table 3.9. FBH Storm Event Breach Parameters

Dam Breach Parameter	FRS No. 1 Breach	FRS No. 13 Breach
Method	Overtop Breach	Overtop Breach
Top Elevation (ft)	382.87	318.91
Bottom Elevation (ft)	347.17	280.11
Bottom Width (ft)	107.1	116.4
Left Slope (H:V)	0.0001	0.0001
Right Slope (H:V)	0.0001	0.0001
Development Time (HR)	0.198	0.216
Trigger Method	Elevation	Elevation
Trigger Elevation (ft)	382.87	318.91
Progression Method	Linear	Linear

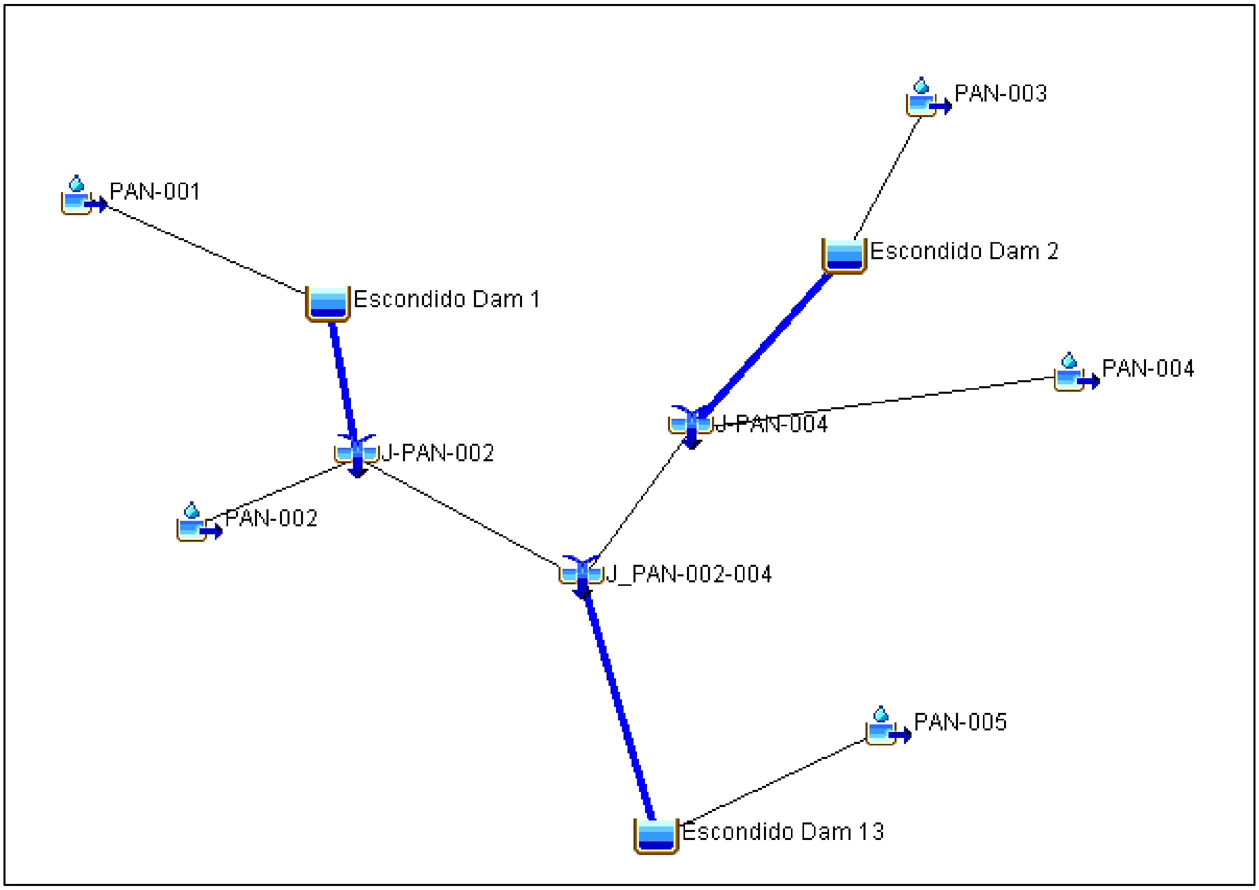

Figure 3.1 HEC-HMS Basin Model Configuration for FBH Breach analysis

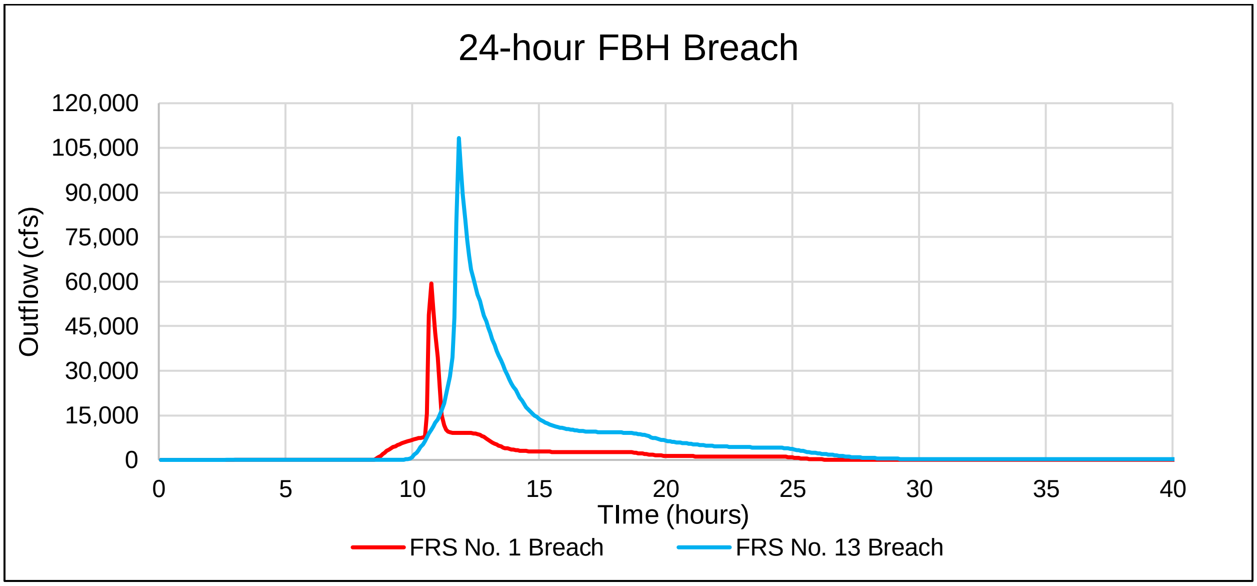

Figure 3.2 24-Hour FBH Breach Hydrograph for FRS No. 1 and FRS No. 13

HEC-RAS 2D 24-hour Breach Modeling

The HEC-RAS 2D model, initially developed for the sunny-day breach analysis, was modified and updated to simulate the 24-hour FBH breach scenarios. Additional breaklines were incorporated and SA/2D connections remained unchanged. The 2D model perimeter was widened and extended further downstream to fully capture the impact of the combined Dam 1 and Dam 13 breach.

The hydraulic model was compiled for two scenarios:

- The HMS hydrograph from the upstream breach of FRS No. 1, was input as an inflow hydrograph below FRS No. 1 and routed to FRS No. 13 to estimate the maximum inundation area and depth between the two dams.
- The HMS hydrograph for the FRS No. 13 breach from the combined HEC-HMS breach evaluation was used as an inflow hydrograph below FRS No. 13 and routed down Panther Creek, Escondido Creek through Kenedy, and to the 2D model limit.

The results of both analyses were then merged into a composite breach inundation boundary using ArcGIS tools. The resulting combined inundation boundary of FRS No. 1 and FRS No. 13 dual breach is provided in **Figure C-6** of the Supplemental Watershed Plan No. III. The PAR was not estimated in detail for this breach scenario, but rather the number of impacted structures above the FFE was estimated via economic analysis. 196 impacted habitable structures were identified, which would result in a PAR greater than 500. This estimate excludes the PAR from the roadway overtopping analysis and the PAR from additional structures in the breach inundation zone not included in the economic analysis. The true PAR would be greater if this additional risk was fully evaluated. This estimate was considered adequate for purpose of showing the risk to the community of a dual dam breach and not refined any further for this study.

3.4 SITES Analysis

3.4.1 SITES Modeling for Existing Condition

The dam hydrologic and hydraulic SITES Integrated Development Environment (SITES) Version 2005.1.12 (USDA, 2022) was used to evaluate erosional stability and head-cutting potential for a vegetated auxiliary spillway channel subjected to flows associated with the design storm events. AECOM has performed a preliminary geologic investigation to evaluate the existing vegetated auxiliary spillway. Four soil borings were collected as part of the geotechnical subsurface investigation: 201-23, 202-23, 203-23, and 204-23. Development of recommended geologic input parameters for SITES analysis was performed according to published NRCS guidance (NRCS 2001, NRCS 2011) and other publications (McCook, 2005).

The SITES parameters recommended for the existing conditions analysis are summarized In **Table 3.10**. Detailed discussion of the analysis assumptions, methodology, and results is provided in **Appendix E-6** of the Supplemental Watershed Plan No. III and Environmental Assessment for the Rehabilitation of Floodwater Retarding Structure No. 1 of the Escondido Creek Watershed, Recommended Geologic Input Parameters for SITES Analysis.

Table 3.10. Recommended Representative Material Parameters for SITES Analysis

SITES Inputs	Proposed Fill (ASW Borrow) (CH)	Upper Clay (CH)	Lower Clay (CL)	Sand and Sandstone (SP)
Plasticity Index (PI)	35	35	20	15
Dry Density (lbs/ft3) – Representative	100	105	110	117
Kh – Representative	0.1	0.3	0.3	0.15
Clay % – Representative	30	30	30	3
Rep. Diam. D75 (mm) – Representative	0.15	0.15	0.1	0.25
Rep. Diam. D75 (in) – Representative	0.00591	0.00591	0.00394	0.00984

The rainfall values used in the FRS No. 1 SITES existing conditions and alternative analysis are provided in **Table 3.11**.

Table 3.11. Escondido FRS No. 1 Rainfall Values for NRCS Design

Storm Event	Source	Rainfall Depth (inches)
50-yr, 24-hour	NOAA Atlas 14, Volume 11, Version 2	9.84
50-yr, 10-day		14.60
100-yr, 6-hour		8.32
100-yr, 24-hour		11.70
100-yr, 10-day		17.00
PMP 6-hr / (FBH)	TCEQ PMP GIS Tool	29.50
PMP 12-hr		35.90
PMP 24-hr / (FBH)		43.10
SDH 6-hr	TR-210-60 Figure 2-2	13.83

The combined 1-day/10-day 100-year Principal Spillway Hydrograph (PSH) run indicates that a peak WSE of 388.30 feet is achieved, assuming the auxiliary spillway does not engage. Since the as-built auxiliary spillway crest is at elevation 377.87 feet, this peak WSE indicates that the auxiliary spillway would engage, which does not meet the NRCS criteria for a high hazard potential structure. The drawdown time after the passage of the PSH was estimated as more than 30 days, which does not meet the NRCS evacuation criteria of passing 85% of the floodpool within 10 days.

The 6-hour Stability Design Hydrograph (SDH) rainfall of 13.83 inches was used to evaluate the stability of the vegetated auxiliary spillway. The stability evaluation was performed following the guidance of Agricultural Handbook #667, Stability Design of Grass-Lined Open Channels (USDA ARS 1987). The auxiliary spillway is considered to have a good vegetation cover with a vegetal retardance curve index of 5.6. The maximum SITES effective soil stress is 3.01 pounds per square foot (psf) and total stress for the existing auxiliary spillway is 4.94 psf. These results exceed the allowable soil stress of 0.187 psf and vegetal stress of 4.20 psf and suggest that soil erosion and sod stripping will probably occur. The current vegetated auxiliary spillway with steepest 10.19% exit slope does not meet the NRCS stability requirements.

The existing auxiliary spillway was evaluated for headcut development and advancement during the 24-hour Freeboard Design Hydrograph (FBH). Preliminary SITES integrity analysis for the existing spillway using the representative soil parameters (i.e., typically between the true average and the lower one-third value of the dataset) indicates that headcutting will occur during the 24-hour freeboard hydrograph (FBH) but is limited to the lower part of the spillway approximately from STA 11+62 to STA 15+78 with an initial spillway slope is 3.2% or steeper. The auxiliary spillway headcutting plot during the 24-hour FBH is presented in **Figure 3.3**.

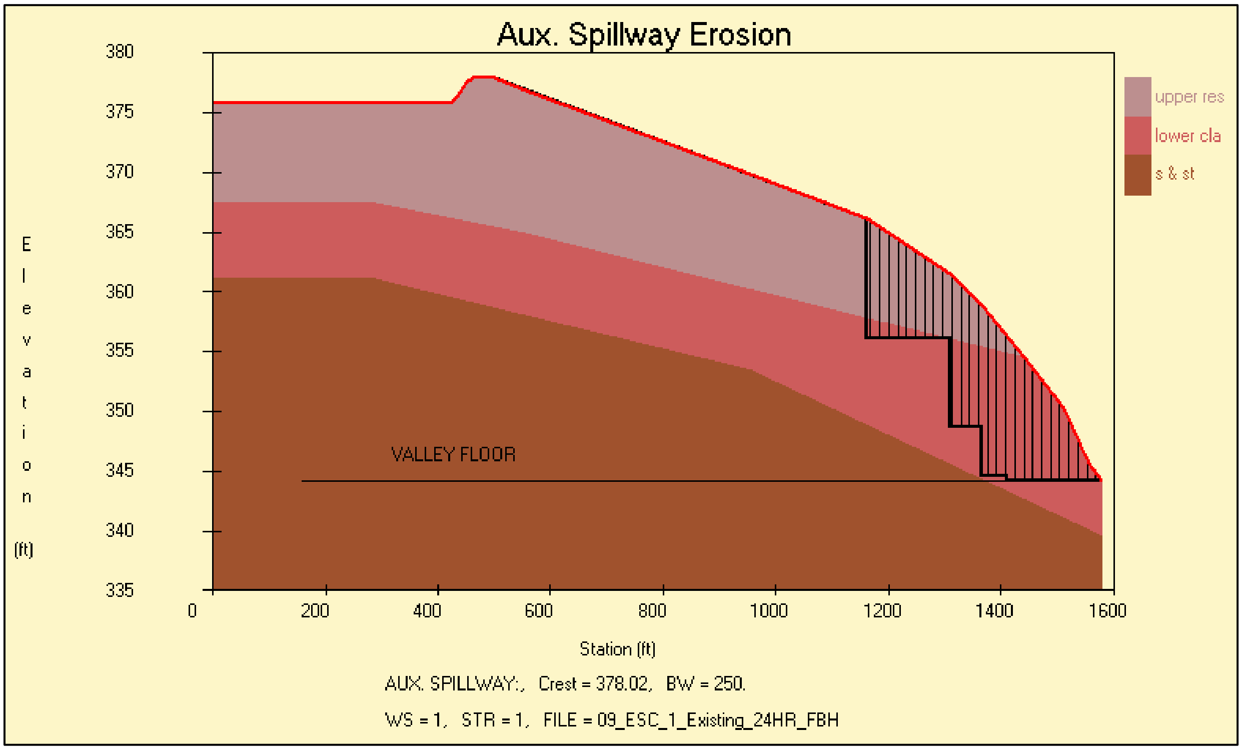

Figure 3.3 Existing Auxiliary Spillway Integrity Analysis Results

3.4.2 SITES Modelling for Alternative 3

The dam hydraulic and hydrologic programs SITES was used to:

- Develop design inflow hydrographs;
- Develop storage-discharge relationships;
- Model the PSH to set the crest of the vegetative auxiliary spillway;
- Model the Stability Design Hydrograph (SDH) and the Freeboard Design Hydrograph (FBH) events;
- Evaluate integrity/stability of the proposed auxiliary spillways;
- Evaluate wave run up height above the SDH peak WSE, and
- Set the top of dam elevation.

The 100-year PSH event was evaluated to select the new size of the principal spillway and set the crest of the vegetative auxiliary spillway. The SITES PSH results are provided in **Table 3.12**. The main goals in sizing this principal spillway system include:

- Safely pass the 1% PSH peak flow with no increase to the existing condition peak 1% PSH flow. The 1% AEP flow is also checked in the HEC-HMS frequency storm analysis.
- Select a crest elevation of the principal spillway riser tower that provides 100 years of future submerged sediment storage (discussed in **Section 2.9**).
- Select a crest elevation of the principal spillway riser tower that allows for standard riser tower hydraulic proportioning (i.e., minimum riser height equal to or greater than three times the pipe diameter).

Two rainfall events evaluated for estimating the peak water surface elevation and setting the top-of-dam crest elevation, including the 6-hour PMP storm with a rainfall depth of 29.50 inches and the 24-hour PMP storm with a rainfall depth of 43.10 inches. The 6-hour PMP storm proved to be the most conservative design storm in setting the top of dam elevation for the high hazard rehabilitation option with a peak water surface elevation of 385.98 feet, rounded to 386.0 feet. The SITES output for Alternative 3 is provided in **Table 3.13**.

Wave setup and wave runup were factored into the analyses in accordance with NRCS procedures (TR-56). The combined wave setup and runup for FRS No. 1 were estimated at 4.0 feet at the SDH peak WSE of 381.92 feet (rounded up to 382 feet). The resulting maximum WSE is 386.0 feet, or 3.13 feet above the existing top of dam elevation of 382.87 feet. The wave runup evaluation results for Alternative 3 are also provided in **Table 3.13**.

The Alternative 3 effective top of dam elevation is 386.0 feet for both evaluations: a) the peak water surface elevation achieved during the FBH evaluation and b) the additional freeboard required for wind and wave action above the SDH elevation.

Table 3.12. Escondido FRS No. 1 SITES PSH Results – Alternative 3

SITES Parameter	100-YR PSH High Hazard 42-in Concept Design
Site Identification	1
Watershed Runoff Curve Number	71
Climatic Index for Karnes County	0.57
Total Watershed Drainage Area (Sq. Miles)	3.22
Watershed Time of Concentration (Hours)	1.39
Initial Reservoir Elevation (Feet)	368.20
PSH Drawdown (Days)	8.67
PS Crest (Feet)	368.20
PS Number of Conduits	1
PS Conduit Diameter (Inches)	42
PS Conduit Area (Sq. Feet)	9.62
Storage, PS Crest (Acre-Ft)	228.8
PS Discharge at AS Crest (CFS)	219.6
AS Crest (Feet)	380.4
Storage, AS Crest (Acre-Ft)	1240.2

SITES Parameter	100-YR PSH High Hazard 42-in Concept Design
Uncontrolled Drainage Area (Sq. Miles)	3.22

Table 3.13. Escondido FRS No. 1 SITES SDH/FBH Results – Alternative 3

SITES Parameter	6-hr SDH/FBH	24-hr FBH
Site Identification	1	1
Watershed Runoff Curve Number	60	60
Total Watershed Drainage Area (Sq. Miles)	3.22	3.22
Watershed Time of Concentration (Hours)	1.39	1.39
SDH Rainfall Total (Inches)	13.83	N/A
SDH Rainfall Duration (Hours)	6	N/A
FBH or Storm Rainfall Total (Inches)	29.50	43.10
FBH or Storm Rainfall Duration (Hours)	6	24
SDH Inflow Peak (CFS)	6963.1	N/A
FBH or Storm Inflow Peak (CFS)	20317.2	9780.3
Initial Reservoir Elevation (Feet)	368.20	368.20
Maximum WS SDH (Feet)	381.92	N/A
Maximum WS FBH or Storm (Feet)	385.98	384.89
Storage at Max. WS FBH or Storm (Acre-Ft)	2147.7	2131.2
Top Dam (Feet)	385.98	385.89
Storage, Top Dam (Acre-Ft)	2148	2132
PSH Drawdown (Days)	N/A	N/A
PS Crest (Feet)	368.20	368.20
PS Conduit Diameter (Inches)	42	42
Storage, PS Crest (Acre-Ft)	229	229
PS Discharge at AS Crest (CFS)	219.6	219.6
PS Discharge for SDH (CFS)	226.3	N/A
PS Discharge FBH or Storm (CFS)	243.1	242.7
AS Crest (Feet)	380.4	380.4
Storage, AS Crest (Acre-Ft)	1240.2	1240.2
AS Width (Feet)	250	250
AS Max. Head SDH (Feet)	1.52	N/A
AS Peak Discharge SDH/Storm (CFS)	790.7	N/A
AS Exit Velocity SDH or Storm (Ft/S)	6.13	N/A
AS Peak Discharge FBH/Storm (CFS)	9393	9175
Wave Run-Up Evaluation		
Effective Fetch (Miles)	0.428	-
Wave Setup (Ft)	0.154	-
Wave Runup (Ft)	3.84	-
Total Residual Freeboard (Ft)	4.0	-
Upper Limit Wave Protection (Ft)	386.0	-

3.4.3 Integrity Analysis Alternative 3

An integrity analysis was performed for the Alternative 3 raised auxiliary spillway parameters using the geotechnical parameters provided in **Table 3.10**. The results of the integrity analysis indicate that the spillway does not breach during the 24-hour FBH using the estimated SITES parameters. The headcut having the maximum final overfall height begins at STA 13+68 and the final height was 8.1 feet. The soil parameters used in the integrity analysis may be re-evaluated during the final design following additional subsurface investigation. The auxiliary spillway headcutting plot for Alternative 3 during the 24-hour FBH is presented in **Figure 3.4**.

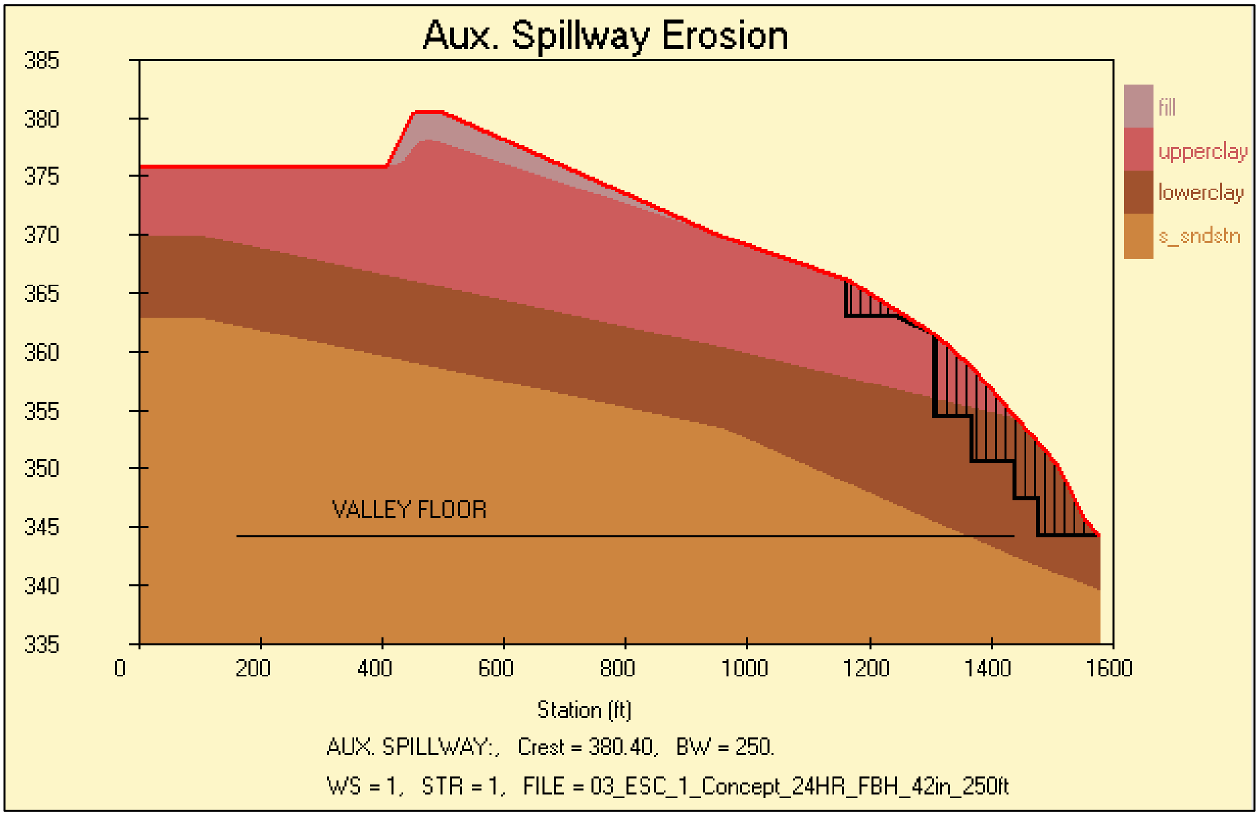

Figure 3.4 Alternative 3 - 24-hour FBH

3.4.4 Stability Analysis Alternative 3

A stability evaluation was performed for the vegetated spillway, following the guidance of Agricultural Handbook #667, Stability Design of Grass-Lined Open Channels (USDA ARS 1987). The evaluation was performed with a vegetal retardance curve index of 5.6.

Using the 6-hour SDH rainfall value of 13.83 inches, the Alternative 3 vegetated spillway was evaluated using the fill material with soil type CH, topsoil specific gravity of 2.65, plasticity index (PI) of 35, and a dry density of 89 lb/ft^3, per **Table 3.10**. The spillway on the raised section will have stable vegetation and soil based upon the concept design raise through STA 13+09, per **Table 3.14**. The spillway passes criteria from STA 5+00 to STA 13+09 with an exit slope of up to of 3.2%. From STA 13+09 to STA 15+78 the spillway does not pass the stability criteria where the slopes range from 4.7% to 10.2%. Below approximate STA 13+10, lining the spillway with an appropriate erosion measure such as articulated concrete block (ACB) would be needed. The lower clay (CL) that daylights at the end of the vegetated spillway was also evaluated for the steeper exit slopes and does not pass stability criteria.

A preliminary evaluation of the Factor of Safety (FOS) for the ACB lining on the spillway exit was conducted based on NEH Part 628 Dams, Chapter 54: Articulated Concrete Block Armored

Spillways (NRCS, 2019). This assessment estimated that the minimum required FOS for FRS No. 1 Alternative 3 is 2.0, given that a failure of the ACB lining would not pose a risk to human life. CONTECH Engineered Solutions, the vendor contacted for the ACB block selection, confirmed the calculated ACB lining FOS for Alternative 3 using a 10.2% slope and peak flow of 9,200 cfs meets the required 2.0 FOS with the ArmorFlex Class 70-T block.

Table 3.14. Alternative 3 Stability Results

STA Range Evaluated	SITES Soil Effective Stress (lb/ft3)	SITES Total Stress (lb/ft3)	SITES Effective Vegetal Stress (lb/ft3)	AH 667 Allowable Soil Stress (lb/ft3)	AH 667 Allowable Vegetal Stress (lb/ft3)	Passes Stability Criteria? (Allowable Stress > Effective Stress)
Proposed Fill - CH						
1162-1309	0.144	1.45	1.31	0.163	4.20	Yes
1309-1368	0.188	1.89	1.70	0.163	4.20	No

3.5 TCEQ Criteria Evaluation

FRS No. 1 falls under the classification of an intermediate size with a high-hazard potential, requiring it to safely accommodate the design flood hydrograph, expressed as a percentage of the Probable Maximum Flood (PMF). As outlined in TAC 299.15(a)(1)(A), the minimum flood hydrograph is determined through a straight-line interpretation within the specified range (from 75% PMF to full PMF), selecting the greater value between the height of the dam or the maximum storage capacity, whichever yields the highest percentage of the PMF. In the case of Escondido FRS No. 1, the design storm was estimated at 76% of the PMF (rounded up from 75.4%), based on a peak storage estimate of 1,746 acre-feet at the effective dam crest elevation.

An average ARC (Type II) curve number of 71.0 was estimated for the contributing Escondido FRS No. 1 subbasin. The Type II curve number (unadjusted) was then converted to a Type III curve number of 84.96 for TCEQ PMF analysis, rounded to 85.0. The PMP rainfall values were obtained using the TCEQ PMP tool (2017) for storm durations 1 through 72 hours and distributed per the temporal distributions presented in Table 4.2 and Figure 4.1 in *Hydrologic and Hydraulic Guidelines for Dams in Texas* (2007). The evaluation of the existing condition indicates that Escondido FRS No. 1 does not meet the TCEQ requirements and does not safely pass the required 76.0% of the PMF. The results of the 76% PMF analysis indicate that the 2-hour PMP event results in both the highest spillway peak discharge and the highest reservoir water surface when compared to the other duration storm events, per **Table 3.15**.

The evaluation of Alternative 3 indicates that Escondido FRS No. 1 will meet and exceed the TCEQ requirements. With the proposed new dam height and increased storage, the required PMF event is 75.7%, rounded to 76% PMF for this analysis. Alternative 3 will safely pass the 76.0% PMF. Unlike the existing condition, where the 2-hour PMP is the governing event, the results for Alternative 3 indicate that the 12-hour PMP is the governing event resulting in both the highest spillway peak discharge and the highest reservoir water surface when compared to the other duration storm events, shown in **Table 3.15**. Note that Alternative 3 can also safely pass up to the 87% PMF as presented in **Table 3.15**. During final design, the TCEQ governing design storm will be used in development of an updated breach inundation map for Escondido FRS No. 1 for future inclusion in a revised Emergency Action Plan.

Table 3.15. Escondido FRS No. 1 Reservoir Routing Results

Storm Duration (hr)	Existing Condition 76% PMF Peak WSE (ft)	Alternative 3 76% PMF Peak WSE (ft)	Alternative 3 87% PMF Peak WSE (ft)
1	380.57	381.59	382.31
2	**383.36**	385.18	385.95
3	383.08	385.11	385.73
6	382.99	384.62	385.28
12	383.13	**385.36**	**385.96**
24	382.53	384.82	385.21
48	381.20	383.52	383.78
72	380.53	382.86	383.07

The **bolded** value indicates the Escondido FRS No. 1 governing TCEQ design storm result.
1. The effective top of dam in the existing condition is 382.87 feet.
2. The effective top of dam for Alternative 3 is 386.0 feet.

4. Roadway Damage Estimation

A total of 11 main road segments (main local roads/state highways) and 28 minor road segments (mostly neighborhood roads) were evaluated for flooding downstream of FRS No. 1 and FRS No. 13 near the City of Kenedy and further downstream (**Figure D-9**). The evaluation was performed during storm events ranging from a 50% to 0.2% AEP and included roadway surface damage, volume of earth fill damage, and guardrail damage based on the flooding depth and extent. The following criteria were used to apply damage repair and debris removal costs to both public and private roads:

- For all roadways, impacts were considered for water depths exceeding 0.5 feet above the lowest deck elevation.
- For main local roads/state highways (i.e. major roads) inundated by 0.5–1.0 feet that are not Low Water Crossings (LWC), a cost of $3,000 is applied for clearing and/or minor repairs.
- For minor roads parallel to Escondido Creek (i.e. local or neighborhood roads) that are inundated more than 0.5 feet, a $3,000 cost is applied for clearing and/or minor repairs for all storm events (i.e. no damages estimated).
- For the three identified LWCs, a $3,000 cost is applied for clearing and/or minor repairs for storm events up to and including the 4% AEP. For storm events with a frequency equal to or higher than the 2% AEP, road damages are assumed to occur as described in the first bullet.
- Repair costs include $18.00 per square yard of inundated asphalt (for resurfacing, a 12-inch subbase, and a 2-inch wearing surface), $30.00 per cubic yard for compacted earthfill, and $200.00 per linear foot for impacted guardrail replacement.

Floodwater damage and debris removal assessments were conducted for each alternative and recurrence interval, as detailed in **Table 4-1**. Three road segments crossing Panther Creek, eight road segments crossing or running parallel to Escondido Creek (including 28 minor local/neighborhood roads were considered for the economic analysis (refer to **Exhibit D-9**). One roadway crossing over Panther Creek and two roadways crossing over Escondido Creek were evaluated as LWCs, as they experience overtopping during smaller storm events, such as those with a 50% or 25% AEP. The damages for the 28 minor roads are aggregated and presented as a combined total damage in **Table 4-1**.

Table 4.1. Road Debris Removal and Repair Cost

Alternative	Total Cost per Recurrence Interval							
	50%	20%	10%	4%	2%	1%	0.5%	0.2%
Panther Creek								
SH 72								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 65,867
Alternative 2	$ 0	$ 0	$ 0	$ 66,385	$ 74,398	$ 71,901	$ 76,891	$ 81,168
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0
CR 163 (LWC)								
Alternative 1	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 88,233	$ 89,455	$ 92,357	$ 95,547
Alternative 2	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 89,262	$ 94,711	$ 98,624	$ 101,894
Alternative 3	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 88,233	$ 89,455	$ 92,342	$ 95,546
Alternative 4	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 88,233	$ 89,455	$ 92,345	$ 95,546
FM 2102								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 56,437
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 50,412	$ 58,016
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 36,225
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 40,307
Escondido Creek								
US 181								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 483,258
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 489,795
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 478,275
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 479,879
N 5th St								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 3,000	$ 352,265
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 311,992	$ 355,233
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 3,000	$ 350,481
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 3,000	$ 3,000	$ 350,420
Helena Rd								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 171,844	$ 228,789
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 191,399	$ 235,678
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 171,844	$ 227,731
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 171,844	$ 226,125
CR 331 (LWC)								
Alternative 1	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 96,948	$ 108,749	$ 115,631	$ 126,443
Alternative 2	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 96,948	$ 108,749	$ 117,399	$ 127,205

Alternative	Total Cost per Recurrence Interval							
	50%	20%	10%	4%	2%	1%	0.5%	0.2%
Alternative 3	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 96,948	$ 108,749	$ 115,631	$ 125,903
Alternative 4	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 96,948	$ 108,749	$ 115,631	$ 126,016
Private Rd (LWC, Evaluated as public road)								
Alternative 1	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 86,793	$ 159,828	$ 246,744	$ 366,590
Alternative 2	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 86,793	$ 159,828	$ 269,900	$ 367,433
Alternative 3	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 86,793	$ 159,828	$ 246,744	$ 366,072
Alternative 4	$ 3,000	$ 3,000	$ 3,000	$ 3,000	$ 86,793	$ 159,828	$ 246,744	$ 366,192
W Main St								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 47,668
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 52,084
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 41,719
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 42,982
SH 72 at Helena Rd								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 32,397
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 32,458
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 31,948
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 28,926
SH 72 East								
Alternative 1	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 62,621
Alternative 2	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 65,308
Alternative 3	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 60,866
Alternative 4	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 61,651
Minor Roads								
Alternative 1	$ 0	$ 0	$ 12,000	$ 39,000	$ 63,000	$ 72,000	$ 84,000	$ 84,000
Alternative 2	$ 0	$ 0	$ 12,000	$ 39,000	$ 63,000	$ 72,000	$ 84,000	$ 84,000
Alternative 3	$ 0	$ 0	$ 12,000	$ 39,000	$ 63,000	$ 72,000	$ 84,000	$ 84,000
Alternative 4	$ 0	$ 0	$ 12,000	$ 39,000	$ 63,000	$ 72,000	$ 84,000	$ 84,000

Alternative 1 – No Action – Dam Remains until Failure
Alternative 2 – Proposed Action – Decommission
Alternative 3 – Proposed Action – High Hazard Rehabilitation: Vegetated Raise
Alternative 4 – Proposed Action – High Hazard Rehabilitation: RCC Stepped Spillway

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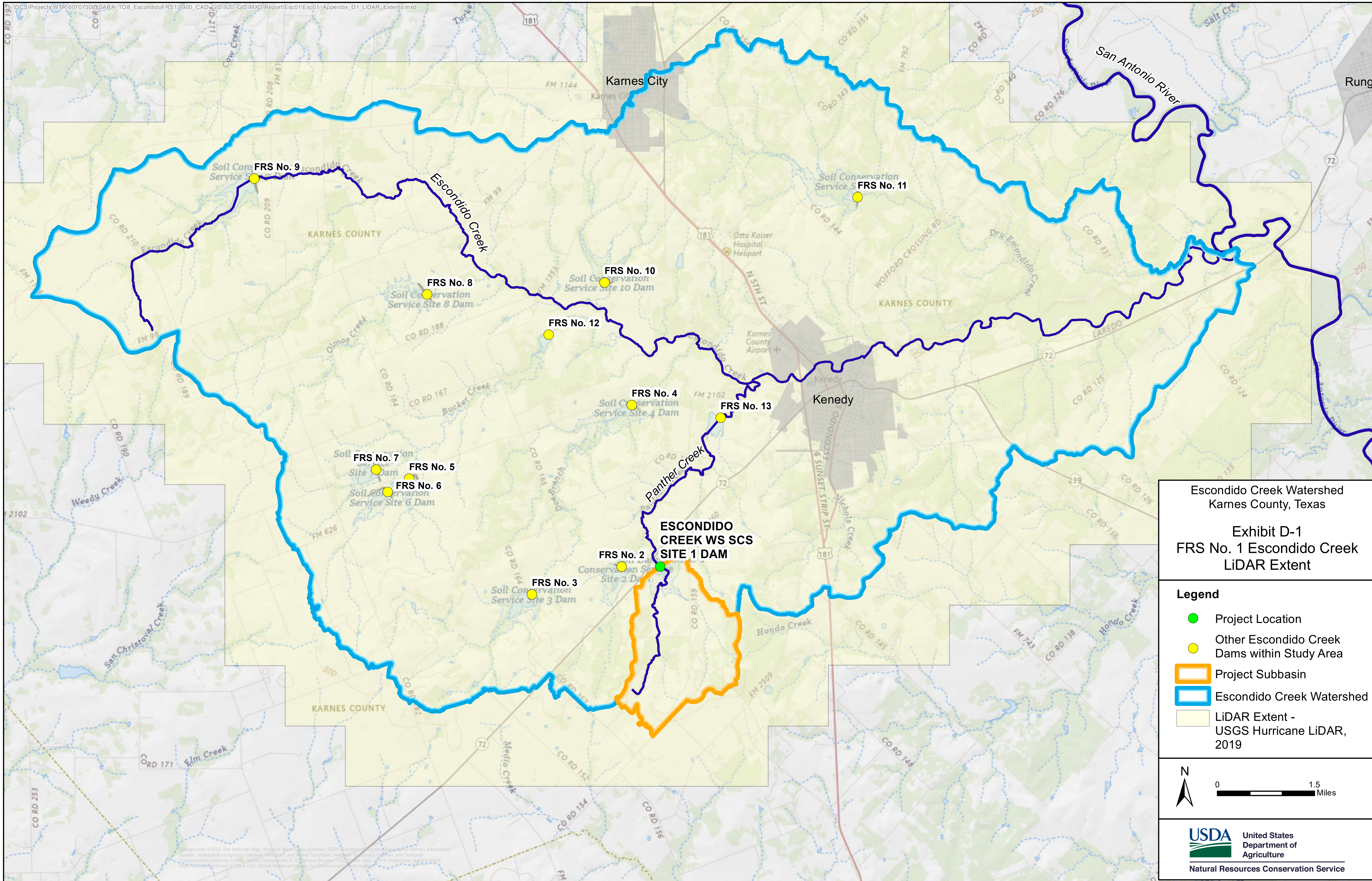
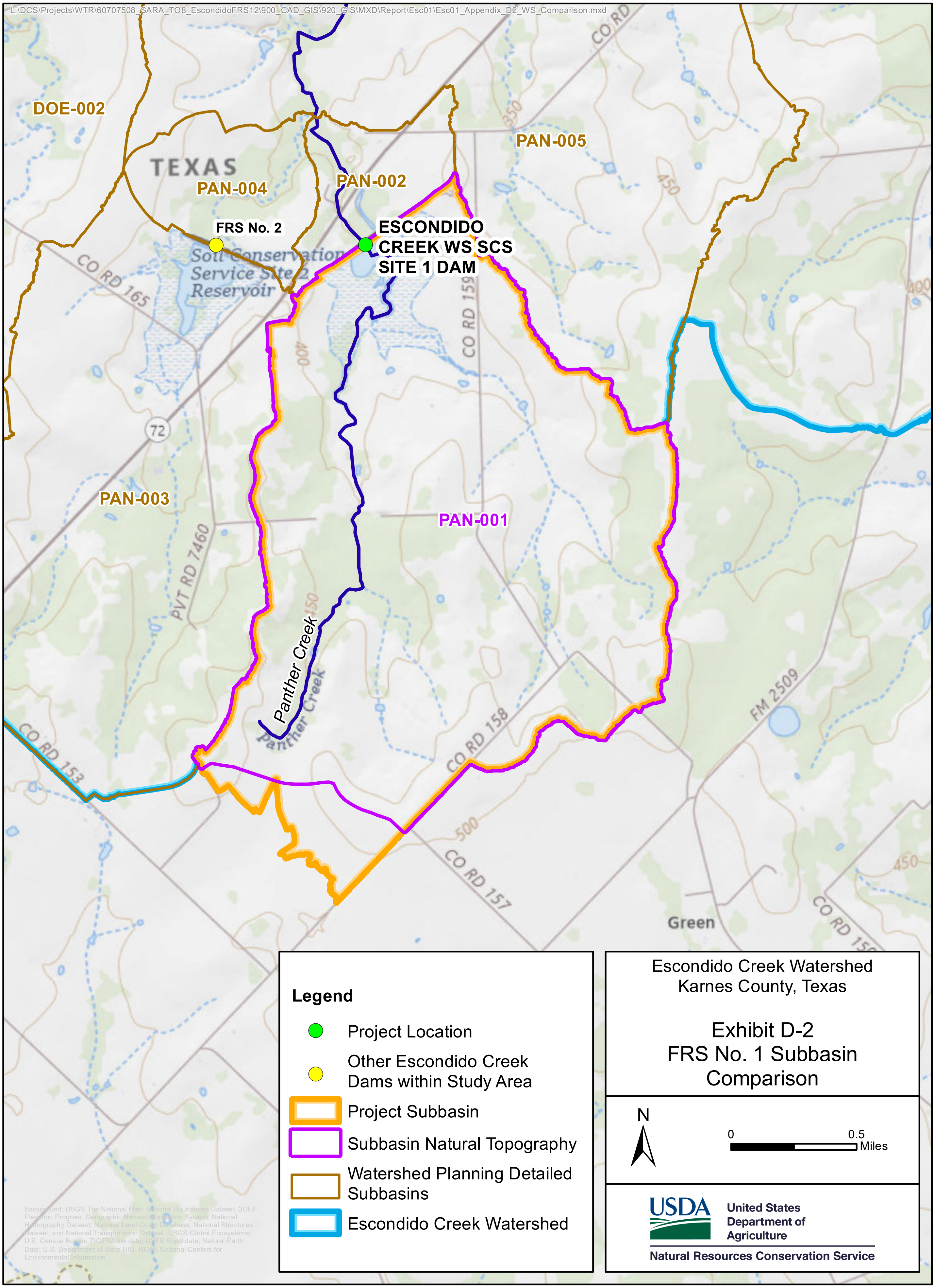
Exhibits



Escondido Creek Watershed
Karnes County, Texas
Exhibit D-1
FRS No. 1 Escondido Creek
LiDAR Extent
Legend
Project Location
Other Escondido Creek Dams within Study Area
Project Subbasin
Escondido Creek Watershed
LiDAR Extent - USGS Hurricane LiDAR, 2019
N
0
1.5
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service
ESCONDIDO CREEK WS SCS SITE 1 DAM
FRS No. 2
FRS No. 3
FRS No. 4
FRS No. 5
FRS No. 6
FRS No. 7
FRS No. 8
FRS No. 9
FRS No. 10
FRS No. 11
FRS No. 12
FRS No. 13
Karnes City
Kenedy
Runge
San Antonio River
Escondido Creek
Panther Creek
KARNES COUNTY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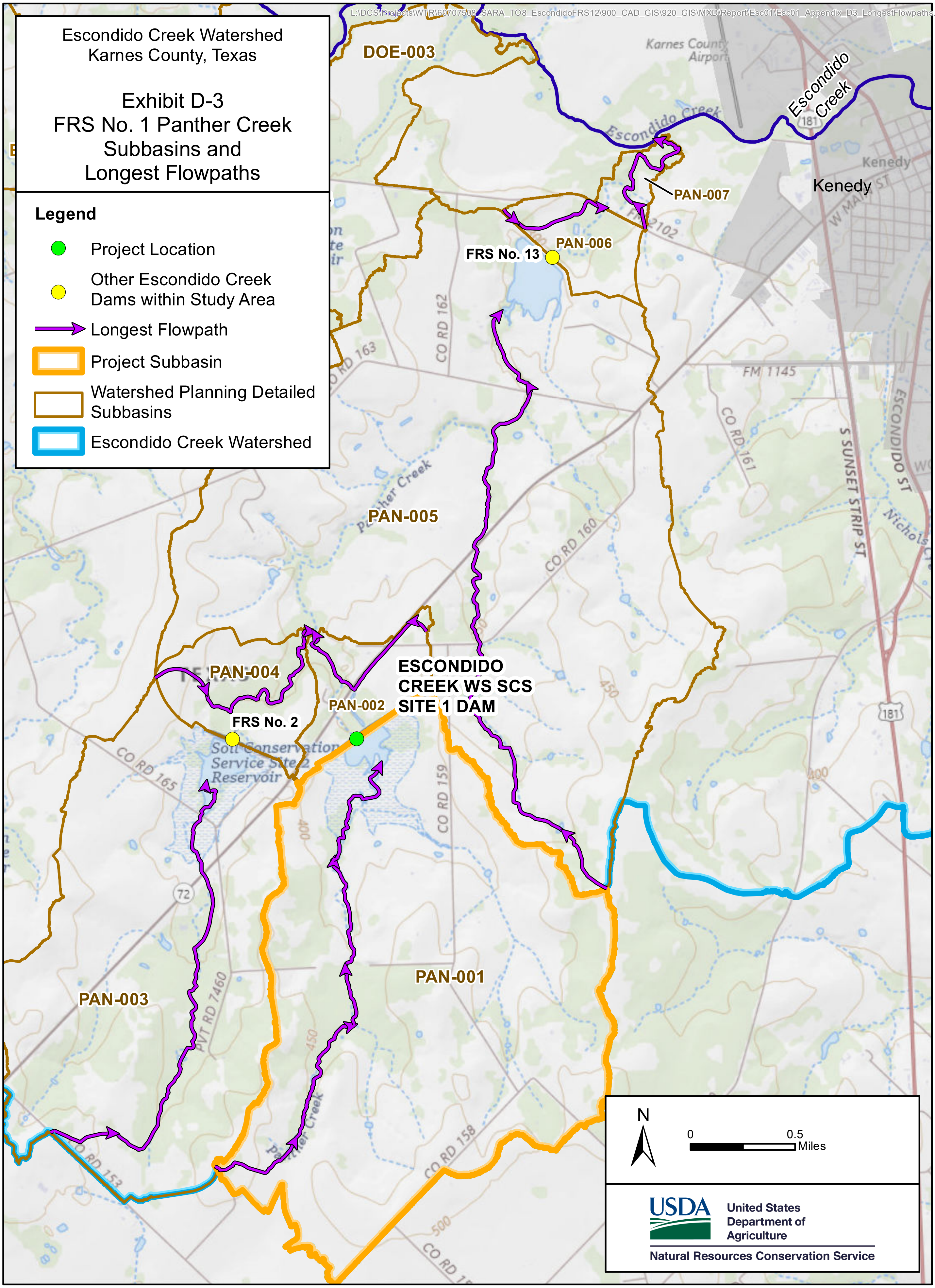
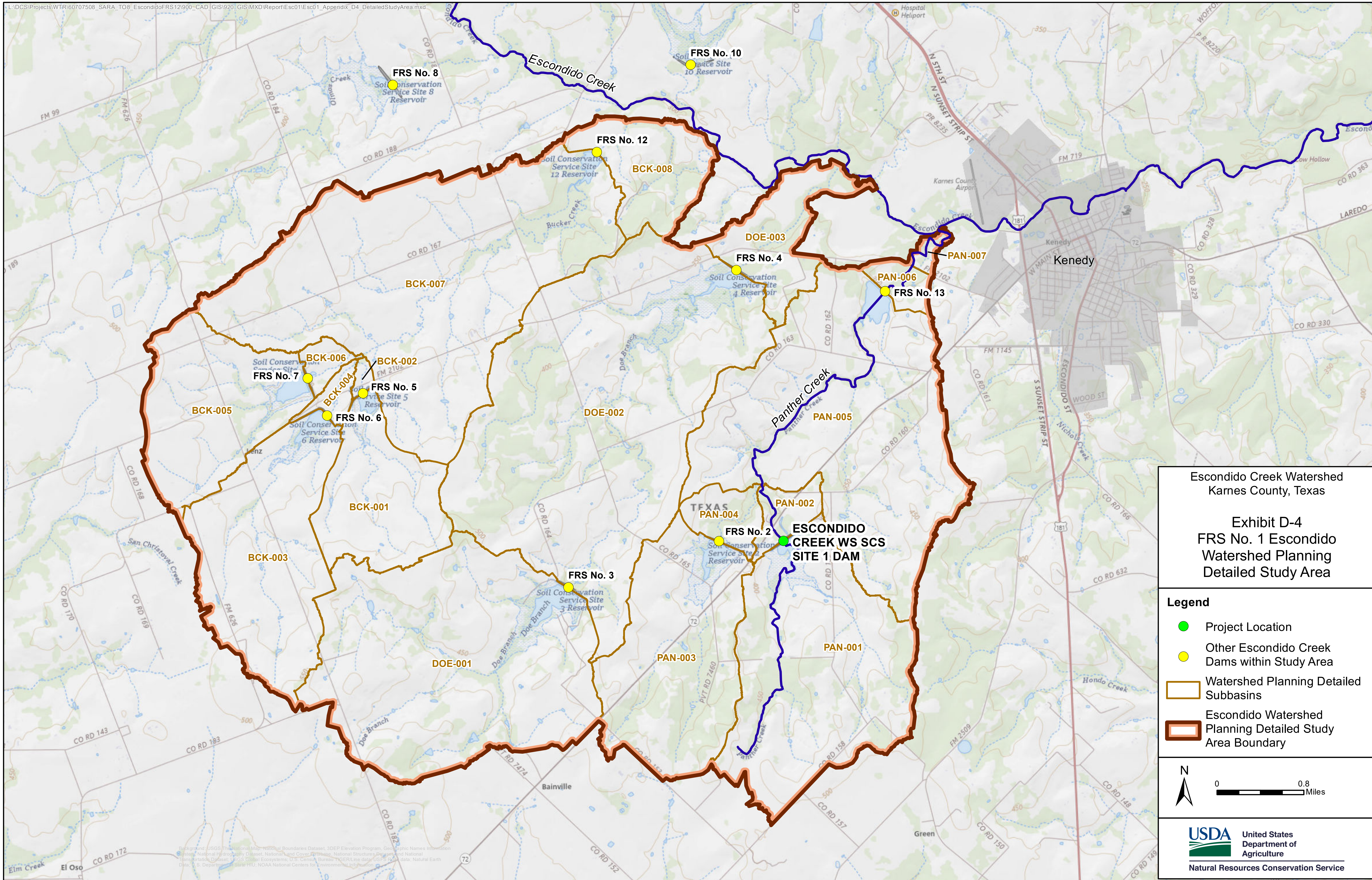
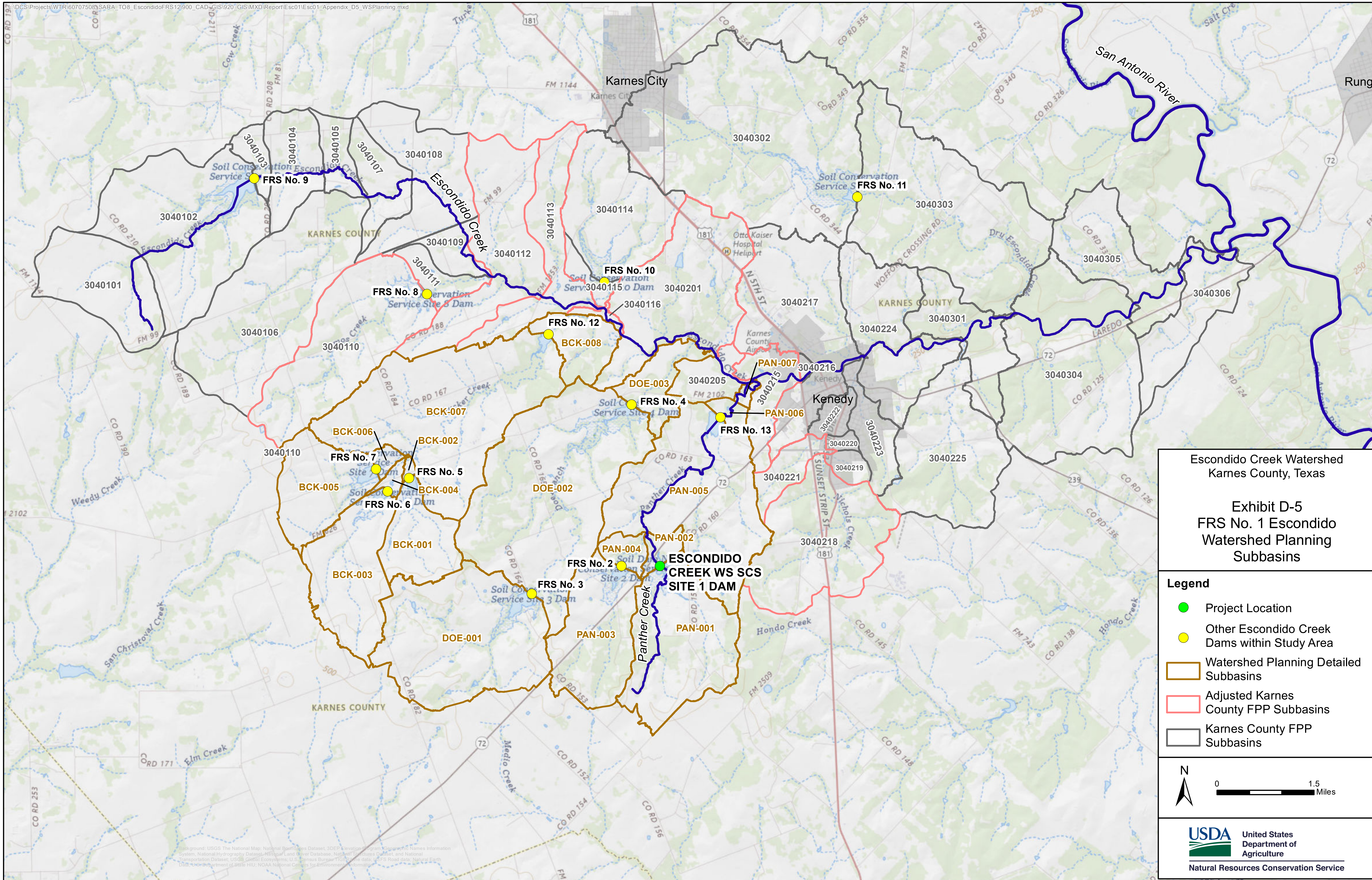
DOE-002
TEXAS
PAN-004
PAN-002
PAN-005
PAN-003
PAN-001
FRS No. 2
ESCONDIDO CREEK WS SCS SITE 1 DAM
Soil Conservation Service Site 2 Reservoir
CO RD 165
CO RD 159
CO RD 158
CO RD 153
CO RD 157
CO RD
PVT RD 7460
FM 2509
72
Panther Creek
Green
Legend
Project Location
Other Escondido Creek Dams within Study Area
Project Subbasin
Subbasin Natural Topography
Watershed Planning Detailed Subbasins
Escondido Creek Watershed
Escondido Creek Watershed
Karnes County, Texas
Exhibit D-2
FRS No. 1 Subbasin Comparison
N
0
0.5
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service



Escondido Creek Watershed
Karnes County, Texas
Exhibit D-3
FRS No. 1 Panther Creek
Subbasins and
Longest Flowpaths
Legend
Project Location
Other Escondido Creek Dams within Study Area
Longest Flowpath
Project Subbasin
Watershed Planning Detailed Subbasins
Escondido Creek Watershed
DOE-003
PAN-007
PAN-006
FRS No. 13
PAN-005
PAN-004
PAN-002
FRS No. 2
ESCONDIDO CREEK WS SCS SITE 1 DAM
PAN-003
PAN-001
Kenedy
Escondido Creek
Karnes County Airport
N
0
0.5
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service



Escondido Creek Watershed
Karnes County, Texas
Exhibit D-4
FRS No. 1 Escondido
Watershed Planning
Detailed Study Area
Legend
Project Location
Other Escondido Creek Dams within Study Area
Watershed Planning Detailed Subbasins
Escondido Watershed Planning Detailed Study Area Boundary
N
0
0.8
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service
ESCONDIDO CREEK WS SCS SITE 1 DAM
FRS No. 2
FRS No. 3
FRS No. 4
FRS No. 5
FRS No. 6
FRS No. 7
FRS No. 8
FRS No. 10
FRS No. 12
FRS No. 13
BCK-001
BCK-002
BCK-003
BCK-004
BCK-005
BCK-006
BCK-007
BCK-008
DOE-001
DOE-002
DOE-003
PAN-001
PAN-002
PAN-003
PAN-004
PAN-005
PAN-006
PAN-007
Escondido Creek
Panther Creek
Kenedy
Bainville


Escondido Creek Watershed
Karnes County, Texas
Exhibit D-5
FRS No. 1 Escondido
Watershed Planning
Subbasins
Legend
Project Location
Other Escondido Creek Dams within Study Area
Watershed Planning Detailed Subbasins
Adjusted Karnes County FPP Subbasins
Karnes County FPP Subbasins
N
0
1.5
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service
ESCONDIDO CREEK WS SCS SITE 1 DAM
FRS No. 2
FRS No. 3
FRS No. 4
FRS No. 5
FRS No. 6
FRS No. 7
FRS No. 8
FRS No. 9
FRS No. 10
FRS No. 11
FRS No. 12
FRS No. 13
Karnes City
Kenedy
Runge
San Antonio River
Escondido Creek
Panther Creek
KARNES COUNTY
BCK-001
BCK-002
BCK-003
BCK-004
BCK-005
BCK-006
BCK-007
BCK-008
DOE-001
DOE-002
DOE-003
PAN-001
PAN-002
PAN-003
PAN-004
PAN-005
PAN-006
PAN-007
3040101
3040102
3040103
3040104
3040105
3040106
3040107
3040108
3040109
3040110
3040111
3040112
3040113
3040114
3040115
3040116
3040201
3040205
3040215
3040216
3040217
3040218
3040219
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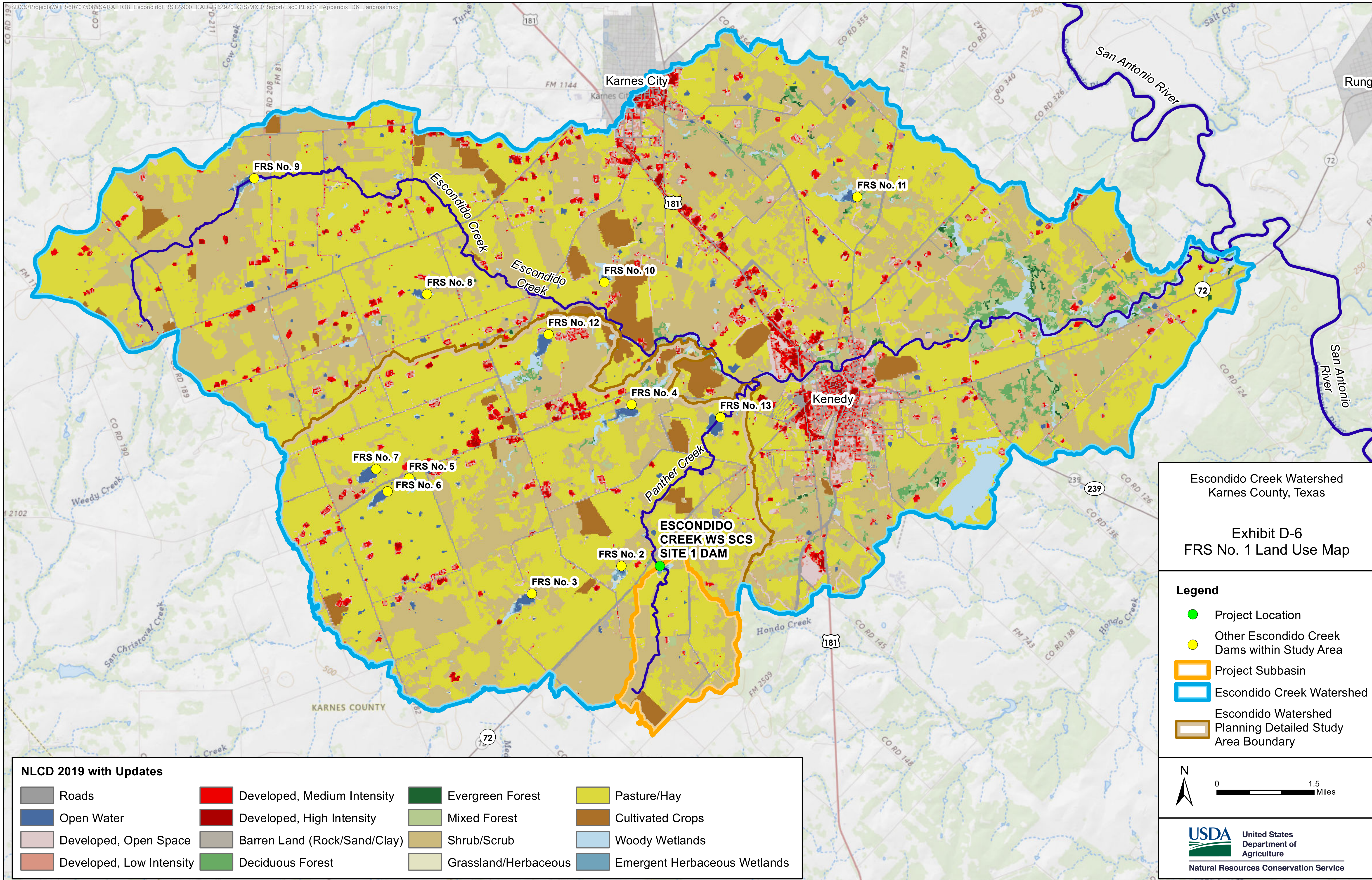
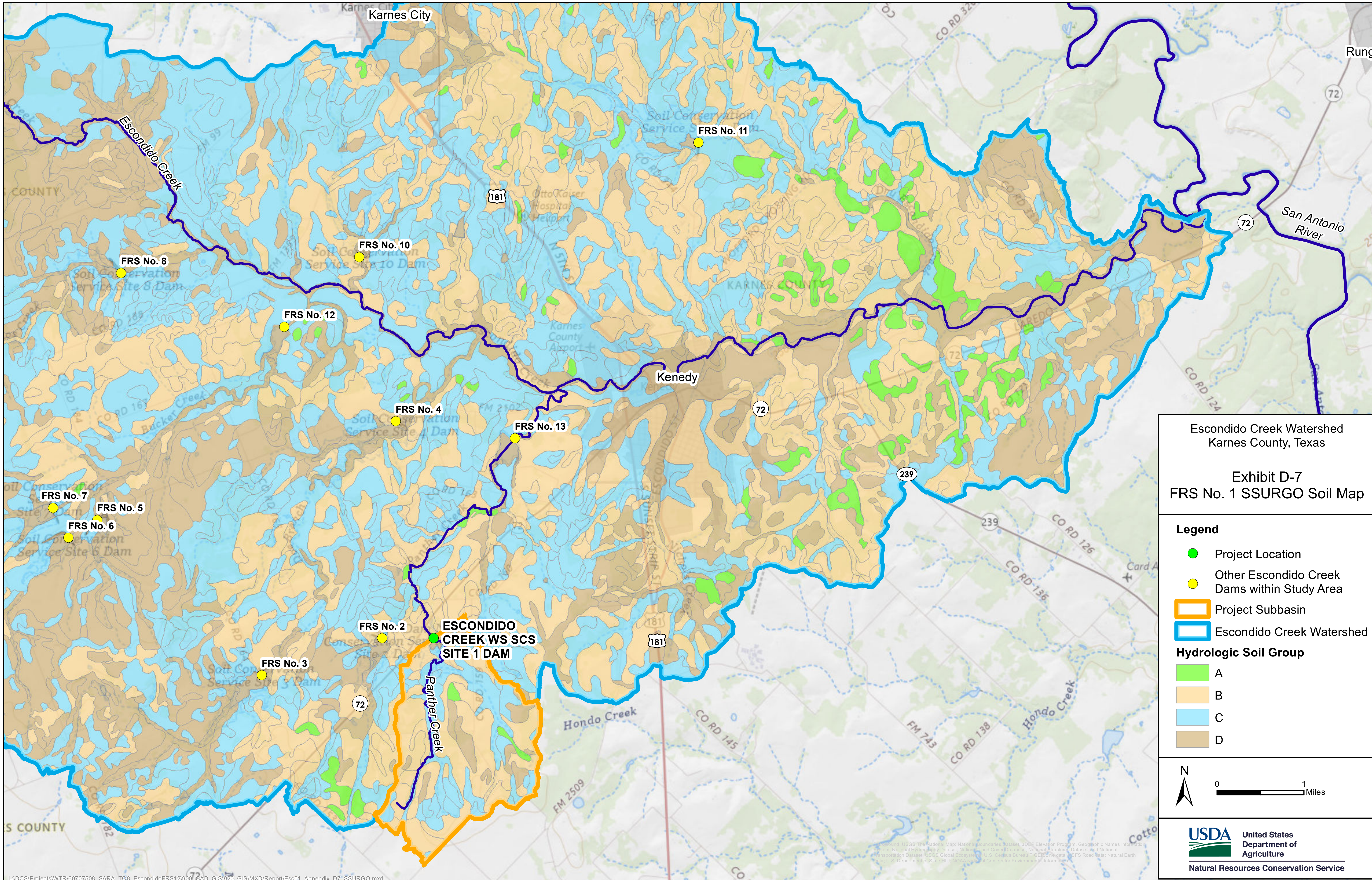
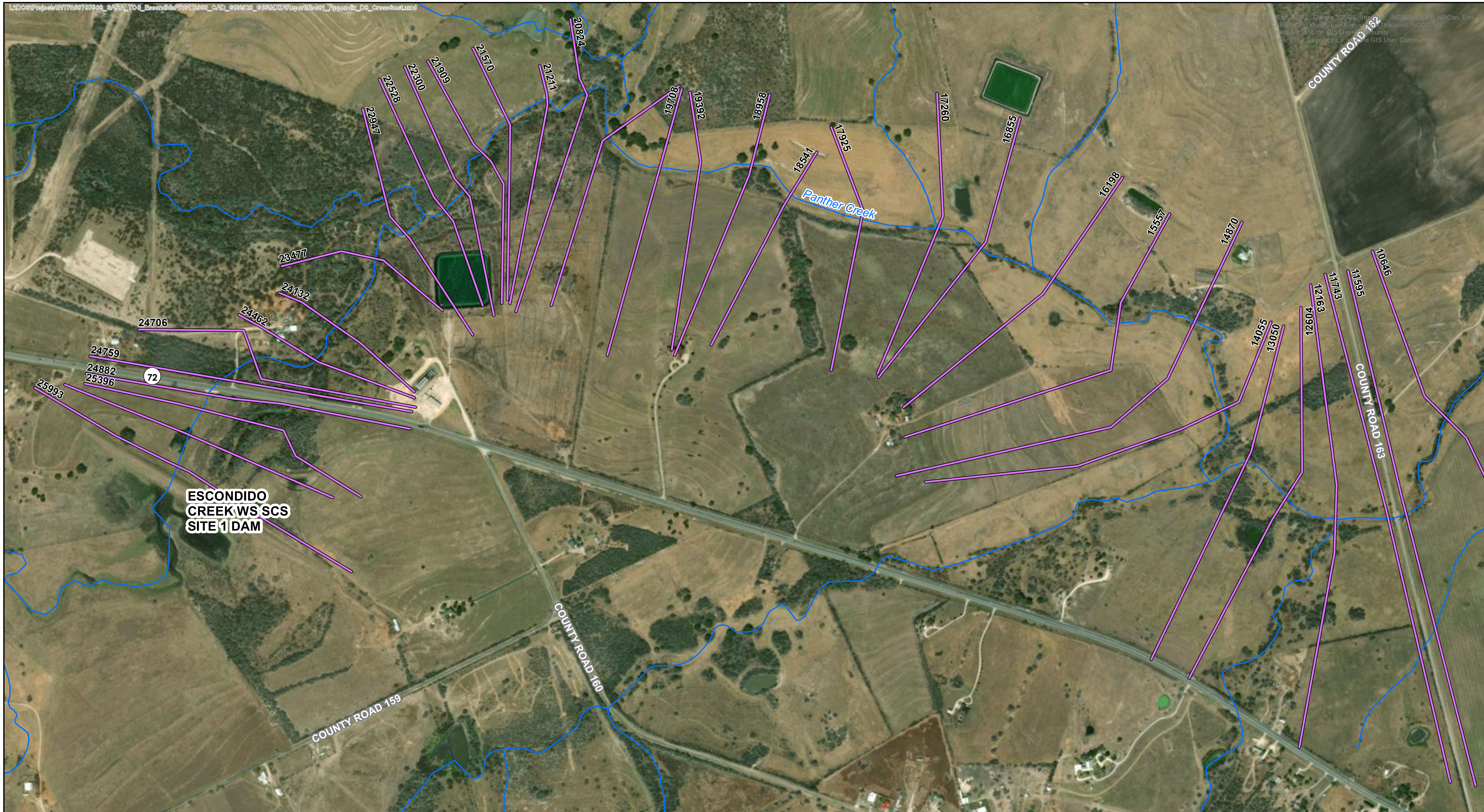
3040304
3040305
3040306

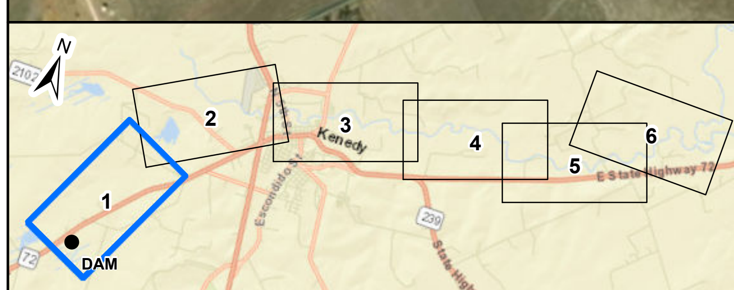


Karnes City
Runge
San Antonio River
Escondido Creek
FRS No. 9
FRS No. 11
FRS No. 10
FRS No. 8
FRS No. 12
FRS No. 4
FRS No. 13
Kenedy
FRS No. 7
FRS No. 5
FRS No. 6
Panther Creek
ESCONDIDO CREEK WS SCS SITE 1 DAM
FRS No. 2
FRS No. 3
Hondo Creek
KARNES COUNTY
Escondido Creek Watershed
Karnes County, Texas
Exhibit D-6
FRS No. 1 Land Use Map
Legend
Project Location
Other Escondido Creek Dams within Study Area
Project Subbasin
Escondido Creek Watershed
Escondido Watershed Planning Detailed Study Area Boundary
NLCD 2019 with Updates
Roads
Open Water
Developed, Open Space
Developed, Low Intensity
Developed, Medium Intensity
Developed, High Intensity
Barren Land (Rock/Sand/Clay)
Deciduous Forest
Evergreen Forest
Mixed Forest
Shrub/Scrub
Grassland/Herbaceous
Pasture/Hay
Cultivated Crops
Woody Wetlands
Emergent Herbaceous Wetlands
N
0
1.5
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service



Escondido Creek Watershed
Karnes County, Texas
Exhibit D-7
FRS No. 1 SSURGO Soil Map
Legend
Project Location
Other Escondido Creek Dams within Study Area
Project Subbasin
Escondido Creek Watershed
Hydrologic Soil Group
A
B
C
D
N
0
1
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service
ESCONDIDO CREEK WS SCS SITE 1 DAM
FRS No. 2
FRS No. 3
FRS No. 4
FRS No. 5
FRS No. 6
FRS No. 7
FRS No. 8
FRS No. 10
FRS No. 11
FRS No. 12
FRS No. 13
Karnes City
Kenedy
Runge
Escondido Creek
Panther Creek
San Antonio River
Hondo Creek
L:\DCS\Projects\WTR\60707508_SARA_TO8_EscondidoFRS12\900_CAD_GIS\920_GIS\MXD\Report\Esc01_Appendix_D7_SSURGO.mxd





0 600 Feet

United States Department of Agriculture

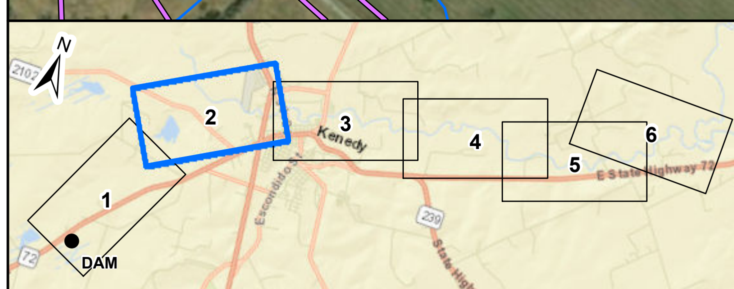
Natural Resources Conservation Service

Legend

- Escondido Creek HEC-RAS Cross-Sections
- Panther Creek HEC-RAS Cross-Sections

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
1 of 6



0 600 Feet

Legend

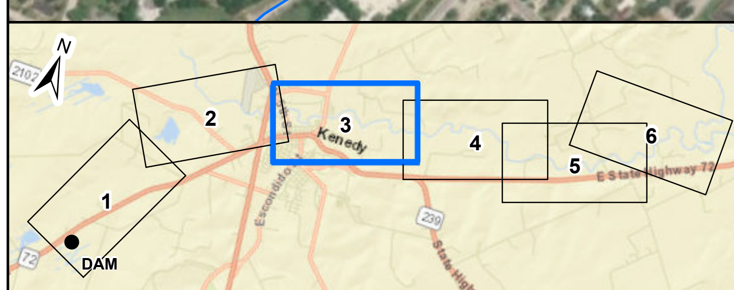
Escondido Creek HEC-RAS Cross-Sections

Panther Creek HEC-RAS Cross-Sections

USDA United States Department of Agriculture
Natural Resources Conservation Service

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
2 of 6



N

0 600 Feet

USDA United States Department of Agriculture

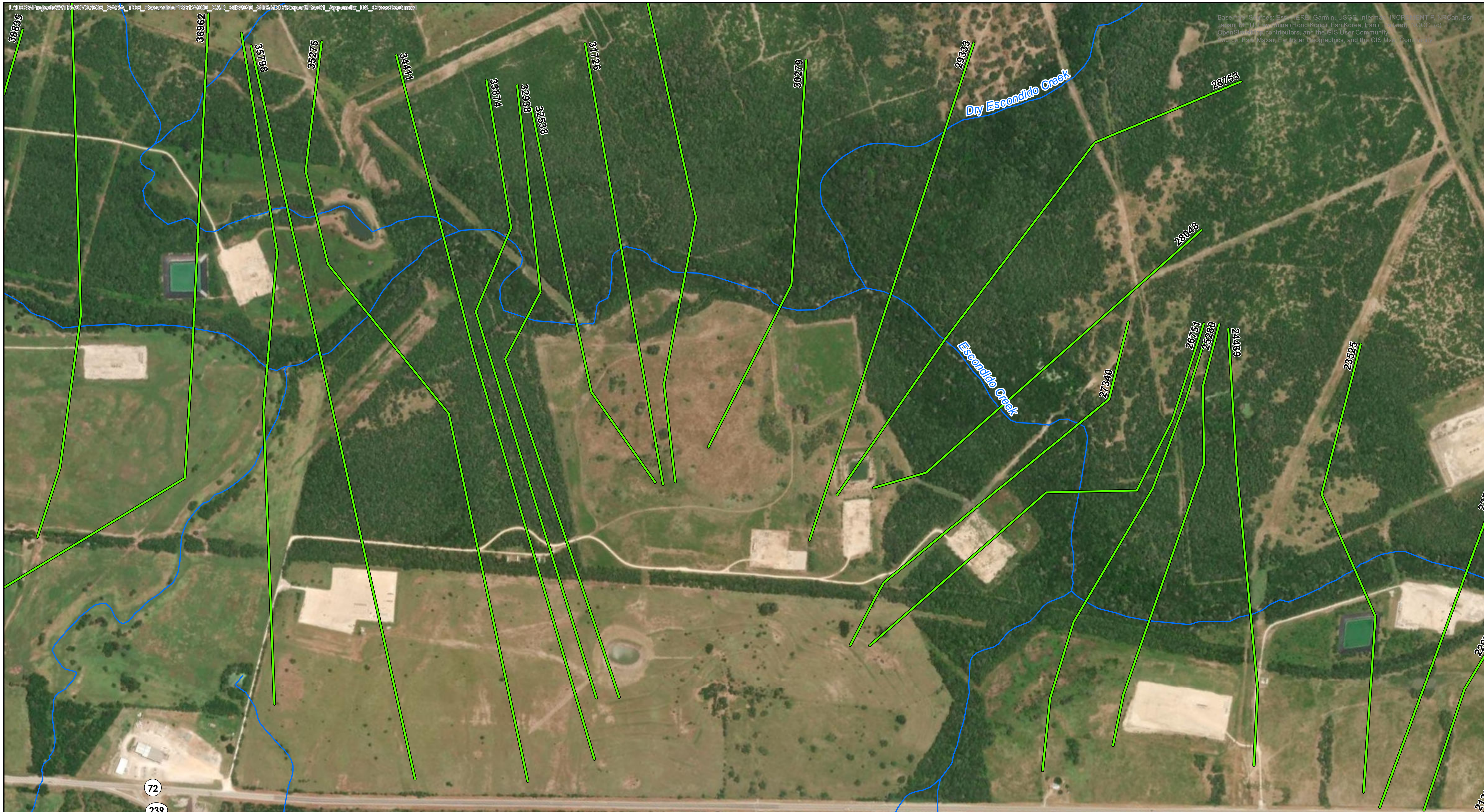
Natural Resources Conservation Service

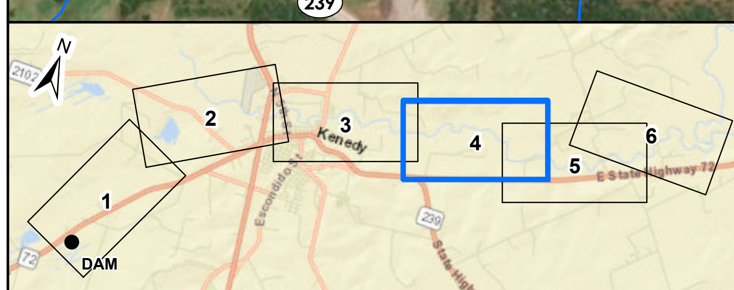
Legend

- Escondido Creek HEC-RAS Cross-Sections
- Panther Creek HEC-RAS Cross-Sections

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
3 of 6





N
0 600 Feet

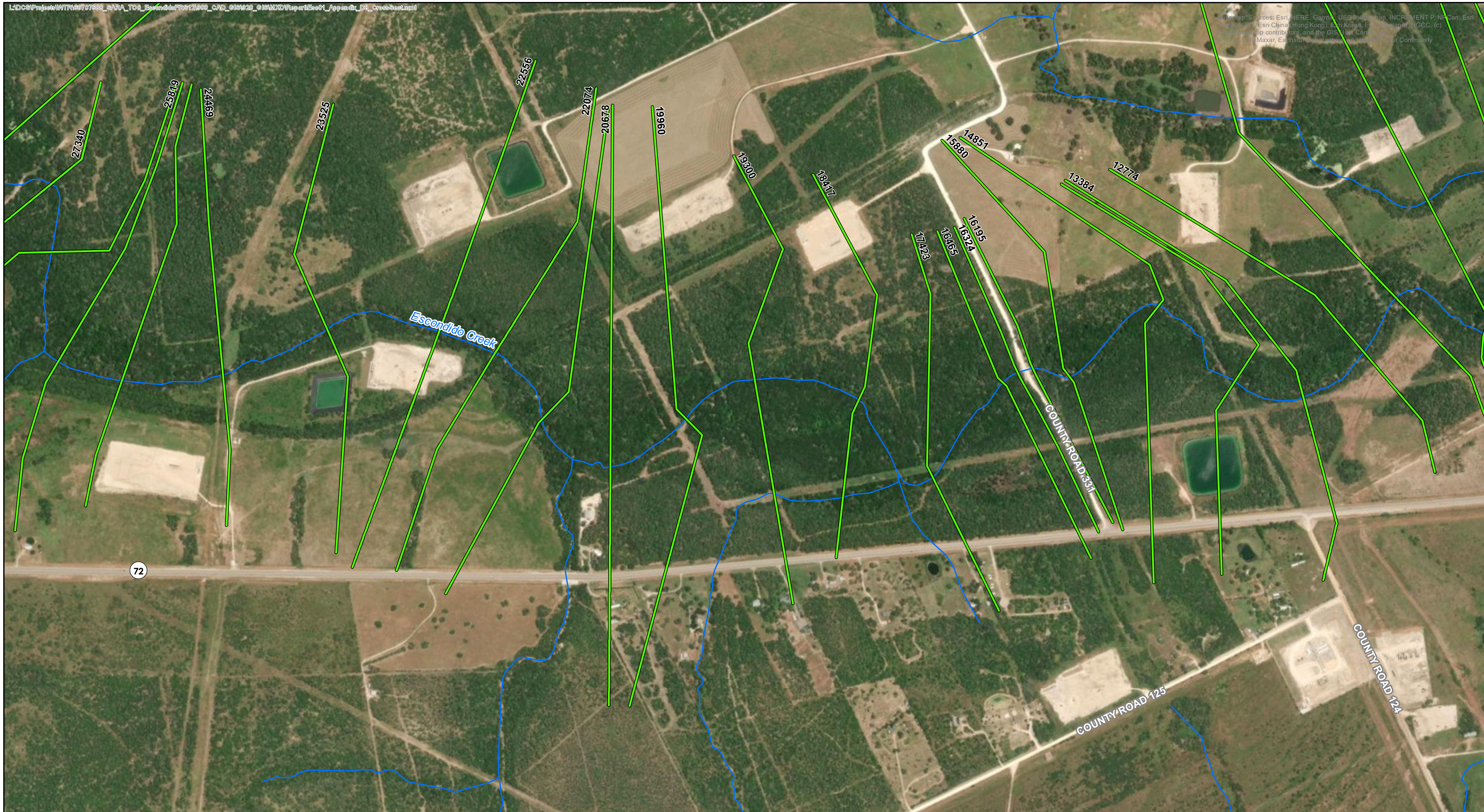
USDA United States Department of Agriculture
Natural Resources Conservation Service

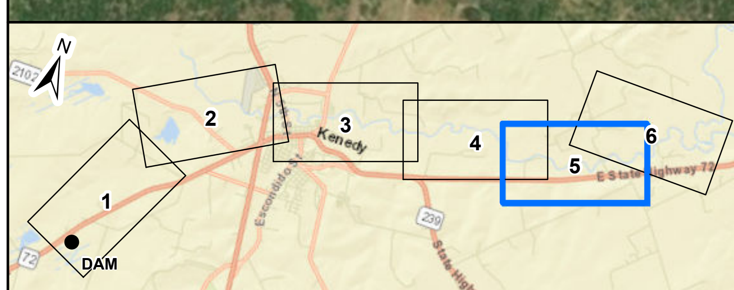
Legend

- Escondido Creek HEC-RAS Cross-Sections
- Panther Creek HEC-RAS Cross-Sections

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
4 of 6





N

0 600 Feet

USDA United States Department of Agriculture

Natural Resources Conservation Service

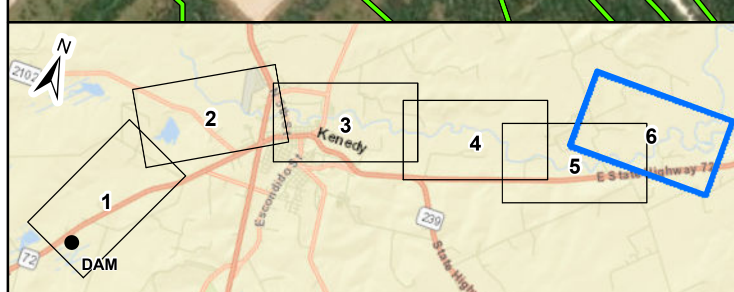
Legend

Escondido Creek HEC-RAS Cross-Sections

Panther Creek HEC-RAS Cross-Sections

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
5 of 6



N

0 600 Feet

United States Department of Agriculture

Natural Resources Conservation Service

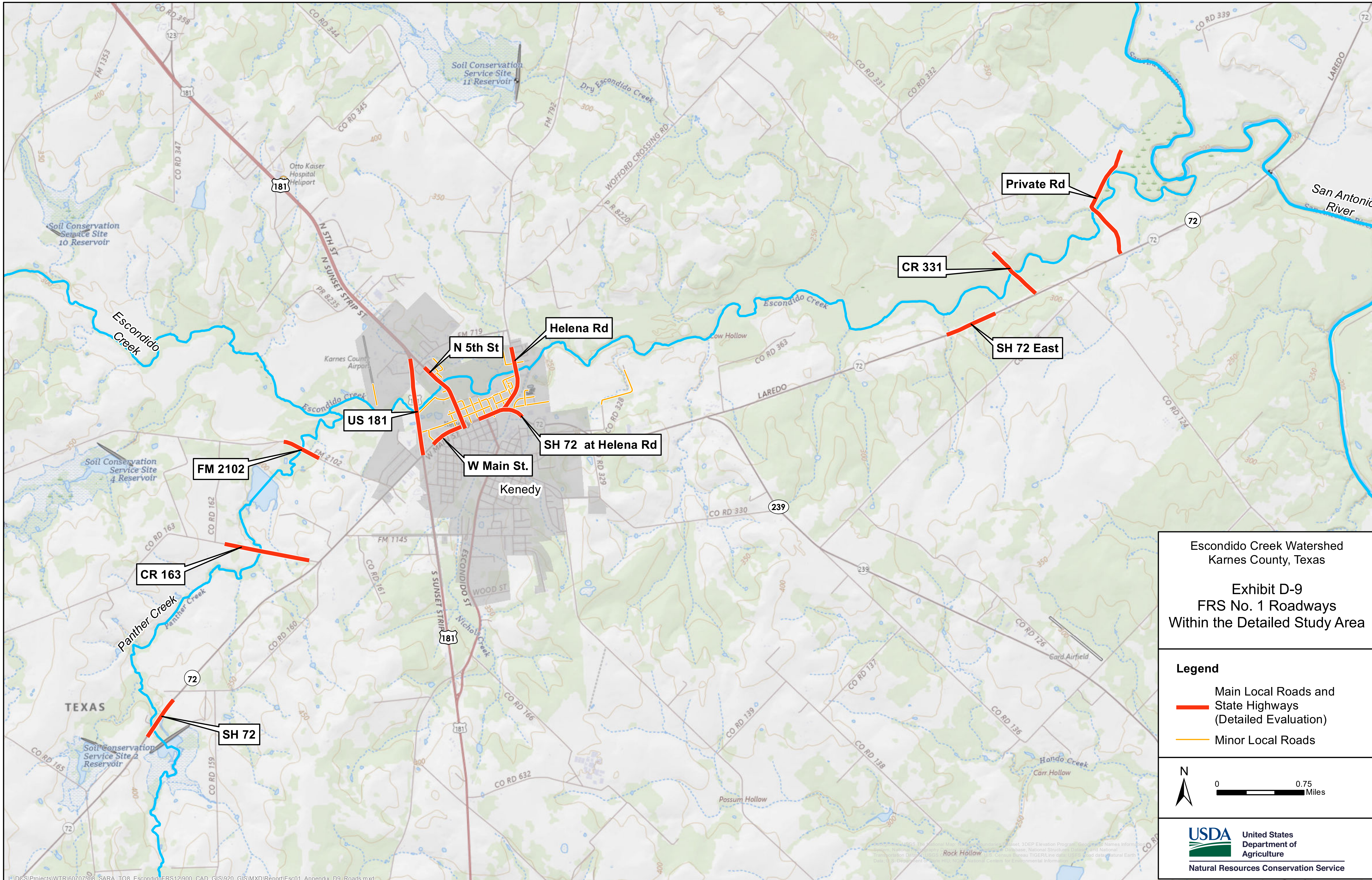
Legend

Escondido Creek HEC-RAS Cross-Sections

Panther Creek HEC-RAS Cross-Sections

Escondido Creek Watershed FRS No. 1
Karnes County, Texas

Figure D-8
FRS No. 1 HEC-RAS
Cross-Section Map
6 of 6



Escondido Creek Watershed
Karnes County, Texas
Exhibit D-9
FRS No. 1 Roadways
Within the Detailed Study Area
Legend
Main Local Roads and State Highways (Detailed Evaluation)
Minor Local Roads
N
0
0.75
Miles
USDA
United States Department of Agriculture
Natural Resources Conservation Service
Private Rd
CR 331
SH 72 East
Helena Rd
N 5th St
US 181
SH 72 at Helena Rd
W Main St.
FM 2102
CR 163
SH 72
Kenedy
Escondido Creek
Panther Creek
San Antonio River
TEXAS
Soil Conservation Service Site 11 Reservoir
Soil Conservation Service Site 10 Reservoir
Soil Conservation Service Site 4 Reservoir
Soil Conservation Service Site 2 Reservoir

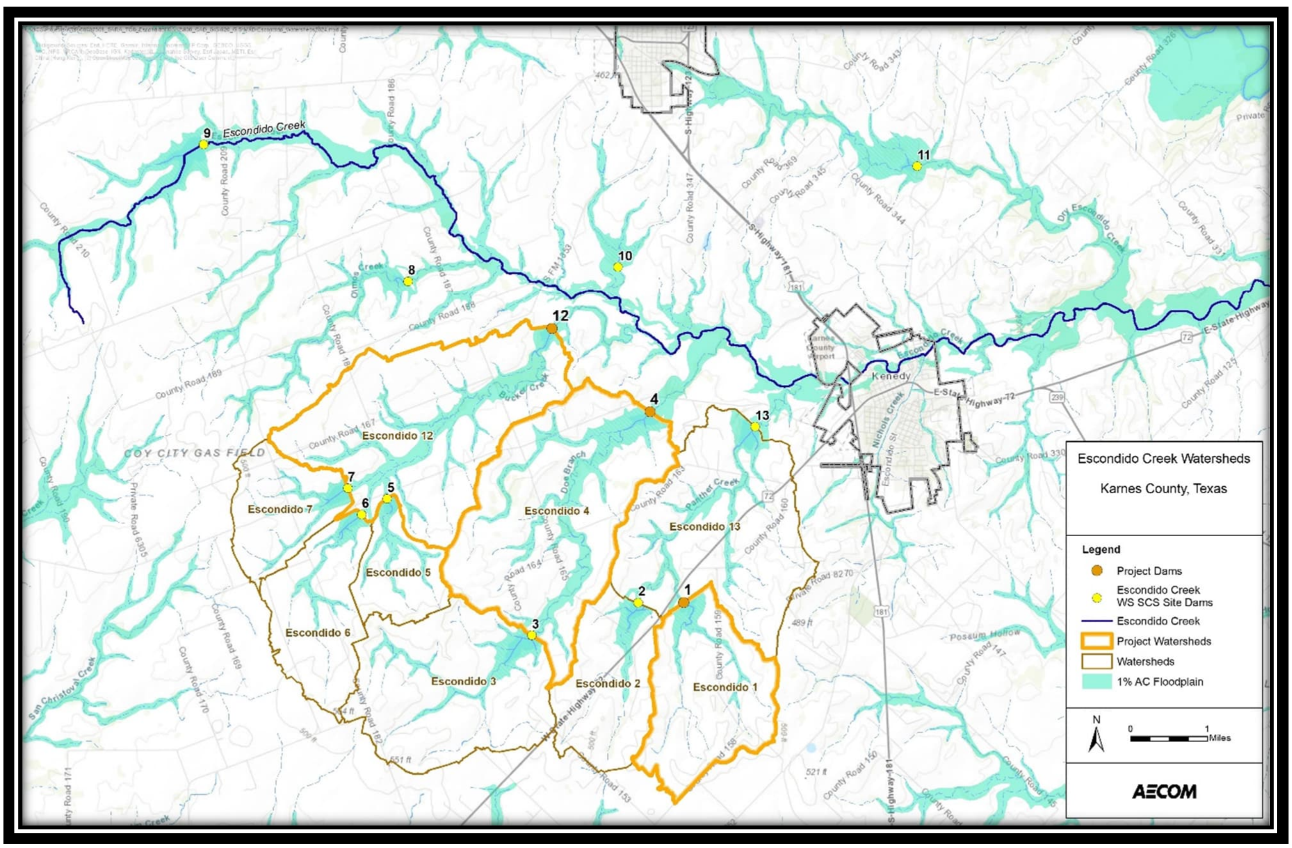
aecom.com

Technical Memorandum – Economic Analysis
Supplemental Watershed Plan and Environmental Document for Escondido FRS No. 1 Supplemental Watershed Plan

D.1 INTRODUCTION

AECOM Technical Services, Inc. (AECOM) conducted economic analyses for four alternatives associated with the Escondido Supplemental Watershed Plan No.1 (the "Project") and Environmental Document. The United States Department of Agriculture, Natural Resources Conservation Service (NRCS), and Karnes County Soil and Water Conservation District, Escondido Watershed District, San Antonio River Authority, and the City of Kenedy as the Project sponsors, propose flood risk management including high hazard rehabilitation potential (HHRP). The Project is located in Karnes County, Texas with the downtown city of Kenedy located to the east. Figure 1 displays the study area, where Dam 1 is on Panther Creek which feeds into the mainstem of Escondido Creek. The NRCS proposes to partially fund the Project through the Watershed Protection and Flood Prevention Act (Public Law [PL] 83-566).

Figure 1. Study Area



Following a preliminary analysis of possible alternatives, four alternatives were carried forward for evaluation. The alternatives are comprised of a No Federal Action (future-without-project [FWOP]) alternative and three future-with-federal-investment (FWFI) alternatives, one of which is federal decommissioning of the dam. **Table 1** describes the alternatives evaluated for the Project. **Table 2** presents the demographic characteristics of the study area on the census tract, city, county, and state level.

Table 1. Description of Project Alternatives

Alternative	Description
Alternative 1. No Federal Action (FWOP)	Continue regular maintenance of the existing system. No modifications would be made to address concerns (i.e., existing conditions would remain).
Alternative 2. Federal Decommissioning (FWFI)	Controlled breach of the dam.
Alternative 3. HHPR (FWFI)	Dam would be rehabilitated to meet both federal and state design standards. 42" conduit and new riser with PS crest at same elevation. Raise existing vegetated auxiliary spillway crest. Install ACB erosion protection and extend cutoff trench below extended embankment. Top of dam raise of approximately 3.1 ft.
Alternative 4. HHPR (FWFI)	Dam would be rehabilitated to meet both federal and state design standards. 42" conduit and new riser with PS crest at same elevation. Raise existing vegetated auxiliary spillway crest. 100 ft wide RCC step spillway and stilling basin. Extend cutoff trench below extended embankment and top of dam raise of 2.7 ft.

Table 2. Demographics of Study Area

Characteristic	Census Tract 9703.02	City Kenedy	County Karnes	State Texas
Population	3,538	3,473	14,710	29,243,342
Median Age	32.8	32.6	33.6	35.2
Median Household Income	N/A	$29,059	$57,798	$72,284
Poverty Rate (all people)	29.8%	30.4%	21%	14%
Unemployment Rate	0.0%	6.4%	3.3%	3.4%

Source: 2018-2022 American Community Survey 5-Year Estimates

D.2 ECONOMIC FRAMEWORK

In general, the national economic benefits and costs presented in this supplemental plan were developed based on guidance contained in the *Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies*[1]. Guidance specific to defining the No Federal

[1] U.S. Water Resources Council, 1983. *Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies*, March 1983.

Action (FWOP) was sourced from the NRCS's *Title 309 – National Instruction, Part 303 – Clarification and Instructions for the No-Action Alternative in Supplemental Watershed Rehabilitation Plans.*[2]

Economic feasibility for a FWFI alternative is determined by comparing the average annual benefits to the average annual costs. The economic analysis considers the No Federal Action alternative as the baseline condition, which assumes the existing conditions with no major changes made to the floodplain. The analysis is formulated from the perspective that changes/impacts resulting from implementation of a FWFI alternative in relation to the No Federal Action alternative were measured as a cost or a benefit (i.e., a zero benefit, zero cost approach was applied to No Federal Action alternative). Costs and benefits are reported in 2024 dollars (2024$) and were evaluated over a 103-year period of analysis (36 months of construction and 100-year evaluation period/design life). The costs and benefits were annualized over the 100-year evaluation period using a 2.75 percent discount rate. Inputs or assumptions provided in a year prior to 2024 were adjusted to 2024 dollars using the U.S. Gross Domestic Product (GDP) deflators.

The hydrologic and hydraulic (H&H) analysis conducted by AECOM for each of the alternatives was used to estimate the depth of flooding throughout the study area. The economic analysis uses inundation models for eight flood recurrence intervals, which are the 50-percent- (2-year), 20-precent- (5-year), 10-percent- (10-year), 4-percent- (25-year), 2-percent- (50-year), 1-percent- (100-year), 0.5-percent- (200-year), and 0.2-percent- (500-year), to estimate future damages from flooding within the study area.

Under the No Federal Action alternative, the dam would not be brought up to current federal or state standards and many of the underlying issues would remain. Therefore, there is a chance for the dam to fail from a seismic, hydraulic, or static event. A static failure was estimated to have the highest probability of occurring. Since a spillway integrity failure does not occur before the static failure, these two failure options were not evaluated as a joint probability or union of events. As a result, a one-time static failure with a probability of 0.01 percent (6,714-year event) for FRS No. 1 was evaluated as part of the No Federal Action alternative. The sunny-day breach only impacts two structures along Panther Creek and therefore has a relatively low value of damages.

D.3 BENEFIT ANALYSIS

The following describes the analyses used to evaluate the benefits of the FWFI alternatives. The benefits represent damage/cost reduction from future flooding and are evaluated in average annual terms. The benefit categories evaluated were:

- Residential and nonresidential structures
- Automobiles
- Debris removal
- Road damages
- Agriculture

D.3.1 Residential and Nonresidential Structures

Knowledge of existing development located in a floodplain is essential when evaluating a flood risk management measure. An inventory was conducted of residential and nonresidential structures located in the study area, which serves as the base data for the economic analysis. The structure inventory comprises residential and nonresidential structures that are within the area of a freeboard hydrograph breach event (the maximum extent of a breach scenario) with a 200 ft buffer. Data from the Karnes County Assessor

[2] USDA Natural Resource Conservation Service, 2022. *Title 309 – National Instruction, Part 303 – Clarification and Instructions for the No-Action Alternative in Supplemental Watershed Rehabilitation Plans,* December 2022.

was obtained, cleaned, and used as the basis for the structure inventory. Detailed descriptions of the data cleaning process can be found in Appendix A. A total of 219 properties were identified based on the data cleaning process performed in GIS.

The structures were assigned a building class and structure type based on the structure descriptions in the Assessor data. **Table 3** lists assigned depth-damage functions, structure types, and number of structures in the inventory. Additionally, the improvement value (excludes land value) listed in the Assessor database was used as a proxy for the depreciated replacement value of the structure.

Table 3. Structure Types in Study Area

DDF	Structure Type	Number of Structures
Fast Food	Non-residential	1
Grocery	Non-residential	1
Hotel	Non-residential	1
Industrial Light	Non-residential	12
Medical Office	Non-residential	1
Mobile Home	Residential – No Basement	43
Non-Fast Food	Non-residential	2
Office One-Story	Non-residential	2
Recreation	Non-residential	2
Religious Facilities	Non-residential	8
Residential-2NB (No Basement)	Residential – No Basement	4
Residential-NB (No Basement)	Residential – No Basement	118
Retail-Clothing	Non-residential	1
Retail-Electronics	Non-residential	8
Warehouse – Non-Refrigerated	Non-residential	15
Total		**219**

The economic analysis was conducted using the USACE, Hydrologic Engineering Center – Flood Damage Reduction Analysis (HEC-FDA)[3] software. H&H data for Escondido and Panther Creek was uploaded into the software and contained river stations and the water surface elevation at each recurrence interval for each of the alternatives. Each structure was assigned to the closest river station using GIS and was formatted and uploaded into HEC-FDA. Each structure was assigned a depth-damage function (DDF) based on the building class. To estimate the depth of inundation in relation to the FFE of each structure, the foundation height was factored into its mean elevation. Structures were assigned a foundation height (height of FFE above the ground) based on the structure type as seen in **Table 4**. The total damages from each recurrence interval were annualized by HEC-FDA to estimate the average annual damages for each alternative. HEC-FDA uses Monte Carlo simulation as part of its risk analysis and is reflected in the final output of the results. This accounts for a level of uncertainty in the economic analysis.

[3] Flood Damage Reduction Analysis (HEC-FDA). https://www.hec.usace.army.mil/software/hec-fda/

Table 4. Assumed Foundation Heights

Structure Type	Foundation Height (Feet Above Ground Level)
Nonresidential	0.5
Residential – No Basement	0.5
Mobile Home	2.5

D.3.2 Automobiles

The damages to automobiles were determined using the USACE EGM 09-04, *Generic Depth-Damage Relationships for Vehicles*.[4] In accordance with the guidance, the elevation of each automobile was assumed to be the mean ground elevation estimated at each structure. The damages to vehicles at residences depends on the following: the average number of vehicles per household and the percentage of vehicles that are likely to be at the residence at the time the flood waters reach the property.

In 2021, the median number of vehicles per household in Karnes County, Texas was two.[5] The average vehicle value was obtained from Consumer Reports[6]. The average retail value for used vehicles was $27,000 in 2023 and was adjusted to 2024 dollars.

The length of potential warning time and the access to a safe evacuation route to a flood-free location were considered to estimate the percentage of vehicles that would likely remain in the flood-prone location. For Karnes County, the analysis assumes that the warning time would be less than 6 hours; therefore, 50.5 percent of the vehicles in the flood area would be evacuated according to USACE EGM 09-04 and 49.5 percent would remain.

Because only those vehicles not used for evacuation can be included in the damage calculations, an adjusted average vehicle value of $27,444 ($27,721 x 2 x 0.495) was assigned to each individual residential structure. The analysis calculated automobile damages for each flood recurrence interval. No automobiles were assigned to nonresidential structures.

D.3.3 Debris Removal

When flooding occurs, debris can accumulate from flood damage, requiring efforts to bring debris to the street for pickup and removal. HEC-FDA does not include this cost in the software, therefore debris removal costs were conducted manually using Excel. The costs associated with debris removal were estimated based on guidance from the Federal Emergency Management Agency (FEMA) and were grouped with structure damages for the purposes of this analysis.

[4] USACE, 2009. *Generic Depth-Damage Relationships for Vehicles*, EGM 09-04. June 22. https://planning.erdc.dren.mil/toolbox/guidance.cfm?Option=BL&BL=OnlyInlandFlood&Type=None&Sort=Default.

[5] Data USA, 2021. *Karnes County, TX.* https://datausa.io/profile/geo/karnes-county-tx

[6] *Consumer Reports, 2023. Used Care Prices Remain High, Making Buying a Challenge.* https://www.consumerreports.org/cars/buying-a-car/when-to-buy-a-used-car-a6584238157/#:~:text=Currently%2C%20the%20average%20price%20of,not%20everyone%20has%20that%20luxury.

Debris removal costs were estimated for every residential structure that incurred flooding above the FFE. The debris costs per structure include the hauling cost, tipping fee, and labor to remove debris and break it into pieces that could be hauled to the street for pickup.

FEMA has estimated there are 25 to 30 cubic yards of debris for a flooded residential structure without a basement and 45 to 50 cubic yards for a residential structure with a basement. The cost to load and haul away debris was estimated using the average cost per cubic yard of $38 from the Homewyse Debris Removal Cost Calculator (October 2023). The FEMA Debris Estimation Field Guide conversion factor of 4 cubic yards of debris per ton was used to convert the total debris removal cost per cubic yard to debris removal cost per ton. In addition, the disposal cost of $20 per ton was included; resultantly, a debris removal cost of $170 per ton was incorporated into the analysis.

Using the Homewyse Debris Removal Cost Calculator (October 2023), the number of labor hours to break down debris and move it from the structure to the street was estimated to be 1.4 hours for every cubic yard of debris. Because homeowners are forgoing other activities to clean up debris, including work and leisure, the opportunity cost was applied to value this time. The value of time was estimated using the 2022 median household income for Karnes County from the Census (adjusted to 2024 dollars) and dividing by 2,080 hours to get $29.13, representing the hourly opportunity cost of work per household. For leisure time, an opportunity cost of $19.42 was assigned based on the common practice used in economics literature to value recreation time as a fraction of the wage. In literature, this fraction ranges from one-third the wage to the full wage; therefore, a fraction of two-thirds was used to estimate the opportunity cost of leisure. During the flood aftermath, owners were assumed to forego recreation two-thirds of the time and forego work one-third of the time, for an average opportunity cost of time of $22.66 per hour. **Table 5** presents the average cost of debris removal from a flooded residential structure without a basement.

Table 5. Summary of Residential Debris Costs – Structure with No Basement

Structure Description	Cubic Yards of Debris	Debris Removal Labor and Disposal Costs	Owner Opportunity Cost of Time	Total Debris Cost
Without Basement	25 to 30	$1,147	$872	$2,020

Note: 2024 price level

D.3.4 Agriculture

Knowledge of existing agricultural land located in a floodplain is essential when evaluating flood risk management measures. Part of the analysis includes estimating the benefits of the FWFI alternatives based on a reduction in agricultural damages. Agricultural impacts assessed include economic losses due to crop damages. The analysis is based on the timing, duration, and extent of flooding. This section describes the methods used to estimate agricultural damages for each alternative.

Flood Impacts

Flood damage to crops can result in replanting, fertilizing, additional spraying, and reduced crop yields. A flood occurring prior to the start of field preparation may cause damages (e.g., reduced yields) from delay of seeding. However, due to uncertainty, these damages were not captured in the model. Following a damaging flood that occurs near the beginning of the growing season, a farmer would likely rework the land and replant the same or a substitute crop to minimize loss. Variable production costs for a replanted crop are usually higher than those without a flood because additional fertilizer must be applied to offset loss of soil fertility. Herbicides are often required to combat weed infestation, and additional preparation of seed beds is required.

Crop loss functions were obtained and used to calculate the weighted average damage per acre of flooded agricultural land. Current normalized prices were used to counteract the frequent short-term fluctuations of crop prices. Total crop damages per acre amount to the difference between the expected net income without a flood and the net income with a flood, accounting for crop damage and additional planting and production costs.

Crops in Study Area

Agricultural Census data was reviewed for the study area in GIS to identify crops grown and their percent of total agricultural land. **Table 6** identifies the crops grown in the study area and the crop distribution.

Table 6. Crops in Study Area

Crop	Percent of Acreage
Forage	39.9%
Corn	10.0%
Sorghum	4.9%
Cotton	44.0%
Oats	1.2%

Crops Progress for Texas Crops

The extent of crop damages from flooding is dependent on the point in the growing season in which a flood occurs. Following a flood that occurs near the beginning of the growing season, it is assumed that a farmer would rework the land and replant the same or a substitute crop to minimize loss. Total crop damages per acre amount to the difference between the expected net income without a flood and the net income with a flood, accounting for crop damage and additional planting and production costs.

Information from the U.S. Department of Agriculture (USDA) was obtained to determine the months in which crops are planted and harvested.[7]

Crop Damage Analysis

The crop progress for Texas crops were used to extract crop DDFs from the USACE, Hydrologic Engineering Center – Flood Impact Analysis software (HEC-FIA). Necessary inputs to obtain loss functions include first plant date, full yield date, last planting date, and end of harvest date. **Table 7** provides definitions for each input, as defined in the *HEC-FIA User's Manual* Version 2.2, and the respective assumption used in the analysis.[8]

[7] USDA, *Field Crops Usual Planting and Harvest Dates, October 2010.* Retrieved https://www.nass.usda.gov/Publications/Todays_Reports/reports/fcdate10.pdf

[8] USACE, 2012. *HEC-FIA User's Manual, Version 2.2*, September. https://www.hec.usace.army.mil/software/hec-fia/documentation/HEC-FIA_22_Users_Manual.pdf.

Table 7. HEC-FIA Crop Assumptions

Date/Days	HEC-FIA Definition	Assumption
First Plant Date	First date that the crop can be planted.	First day of the earliest month in which crops were planted in Texas.
Full Yield Date	Latest date that the crop can be planted and still reach full maturity.	One month prior to the Last Planting date.
Last Planting Date	Latest date that the crop can be planted with a reduction in yield.	Last day of latest month in which crops were planted in Texas.
End of Harvest Date	Last date that the crop is harvested, after which little damage can be incurred.	Last day of final month in which crops are harvested.
Dryout Period (days)	Number of days after a flood has receded before the soil is sufficiently dry so replanting can begin. FIA automatically adds 7 days on to the dryout period to allow for re-cultivation of the fields.	The default value in HEC-FIA. Default includes a period of 3 days for dryout and 7 days for recultivation for a 10-day period following inundation before crops can be replanted.

The data on crop progress was used to make the crop planting and harvesting date assumptions shown in **Table 8**, which were then used to extract the appropriate DDFs from HEC-FIA.

Table 8. Crop Plant Data Assumptions

Crop	First Plant Date	Full Yield Date	Last Planting Date	End of Harvest Date
Forage	1-May	15-May	1-Jun	25-Sep
Corn	1-Mar	7-Apr	17-May	8-Nov
Sorghum	1-Mar	1-May	5-Jul	6-Dec
Cotton	22-Mar	7-May	20-Jun	11-Jan
Oats	7-Sep	14-Oct	25-Nov	4-Jul

The HEC-FIA loss functions estimate “Initial % Crop Loss” values, which represents an estimate of the percentage of the mature crop value that is expected to be damaged if the crop was planted on the First Plant Date. Initial % Crop Loss values depend on the date in which a flood event occurs, and factors crop plant data into its estimates. HEC-FIA also categorizes crop damages as a percent of crop yield for flood events lasting 0, 3, 7, and 14 days, based on the vulnerability of the crop to flood damage. Straight line interpolation was used to estimate total crop damages as a percent of crop yield for flood events lasting 1 to 2 days.

Tables 9 and 10 provide the crop damage assumptions for 1 to 2 days of inundation.

Table 9. Crop Damages from 1 Day Inundation

Crop	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Forage	12%	13%	16%	18%	20%	24%	29%	32%	21%	10%	10%	11%
Corn	0%	0%	0%	4%	13%	22%	25%	27%	32%	24%	10%	1%
Sorghum	0%	0%	0%	4%	13%	22%	25%	27%	32%	24%	10%	1%
Cotton	7%	7%	8%	8%	11%	17%	20%	20%	21%	20%	10%	10%
Oats	0%	0%	0%	4%	13%	22%	25%	27%	32%	24%	10%	1%

Table 10. Crop Damages from 2 Days Inundation

Crop	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Forage	25%	27%	31%	36%	40%	47%	57%	64%	42%	19%	19%	22%
Corn	0%	0%	0%	8%	27%	44%	50%	55%	64%	48%	20%	2%
Sorghum	0%	0%	0%	8%	27%	44%	50%	55%	64%	48%	20%	2%
Cotton	14%	15%	16%	16%	21%	35%	40%	41%	43%	40%	20%	20%
Oats	0%	0%	0%	8%	27%	44%	50%	55%	64%	48%	20%	2%

Crop Planting and Production Prices

To monetize crop damages, the analysis estimated planting costs and production costs for each of the crops, estimated by acre, using data on budgets by commodity sourced from USDA and UC Davis Cooperative Extension Sample Costs to Produce. Additional planting costs per acre were seed, equipment operating expenses, and crop insurance. Additional production costs were herbicides, fertilizer, fungicide, and insecticide. **Table 11** lists estimates for crop replanting and production costs per acre for primary crops in the study area.

Table 11. Planting and Production Costs per Acre

Item	Crop				
	Forage	Corn	Sorghum	Cotton	Oats
Replanting Costs (per acre)	$170	$249	$87	$353	$88
Production Costs (per acre)	$221	$412	$412	$601	$112

Sources: UC Davis and USDA.

Values listed in **Table 11** were used to estimate replanting costs per acre following a flood event. The analysis assumes replanting would occur following any flood event that occurred in a month in which the damaged crop could be replanted (i.e., before the last plant date). Acres replanted were assumed to equal acres damaged in the flood event.

Crop Production Value

To complete the assessment of agricultural flood damages, production values of each crop per acre were estimated by multiplying the average yield by the normalized price per unit. Yields per acre were obtained from USDA National Agricultural Statistics Service.[9]

Normalized prices for all crops were also sourced from USDA National Agricultural Statistics Service. **Table 12** displays the average crop yield and average production value per acre for all crops in the analysis.

Table 12. Average Crop Yield and Average Production Value

Item	Crop				
	Forage	Corn	Sorghum	Cotton	Oats
Average Yield (unit/acre)	2 tons	95 bu	53 bu	734 lbs	54 bu
Normalized Prices (per unit)	$198	$8	$5	$1	$7
Average Production Value per Acre	$339	$747	$264	$811	$352

Sources: USDA

Likelihood of Flooding by Month

To determine the likelihood of a flood occurring each month, the probability that a storm event would occur in a month was estimated from monthly precipitation data for Karnes County between 1990 and 2022 from the National Oceanic and Atmospheric Administration's (NOAA's) National Center for Environmental Information. Average monthly precipitation values were divided by the total average annual precipitation to calculate the percentage of precipitation that occurs each month. The percentage of precipitation per month was used as a proxy for the likelihood of a storm event occurring, as displayed in **Table 13**.

Table 13. Likelihood of Flooding by Month

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2.1%	1.8%	2.7%	2.9%	4.5%	3.8%	3.5%	2.8%	4.4%	3.4%	2.5%	2.2%

Source: NOAA National Centers for Environmental information, Climate at a Glance: County Time Series, retrieved October 2023 from https://www.ncdc.noaa.gov/cag/

Agricultural Benefits

An Excel based model was developed to estimate the average annual agricultural damages for the No Federal Action and FWFI alternatives. The model incorporates the factors presented above and the H&H

[9] USDA National Agricultural Statistics Service – Quick Stats. https://quickstats.nass.usda.gov/

data that estimates the agricultural acres flooded by duration of either 24 or 48 hours for six flood recurrence intervals (0.2%, 0.5%, 1%, 2%, 4% and 10%).

A summary of agricultural benefits by project alternative is provided in **Table 14**.

Table 14. Summary of Agricultural Benefits

Alternative	Annual Damages	Annual Benefits
Alternative 1- No Federal Action	$3,000	$0
Alternative 2 – Federal Decommission	$3,000	$0
Alternative 3 - HHPR	$3,000	$0
Alternative 4 - HHPR	$3,000	$0

*All values are rounded

D.3.5 Benefit Summary

This section summarizes the benefits analysis, which includes comparisons of the impacts to structures from the alternatives. Structure-related benefits include damage reductions to structures, contents, automobiles, and debris removal. A summary of damages for all alternatives by recurrence interval is provided in **Table 15.**

Table 15. Summary of Damages by Recurrence Interval (2024$)

Recurrence Interval		Building & Autos	Contents	Road Damages	Debris Removal	Total Damages
Alternative 1 – No Action						
50%	2-year	$1,000	$1,000	$9,000	$0	$11,000
20%	5-year	$15,000	$8,000	$9,000	$2,000	$35,000
10%	10-year	$69,000	$27,000	$22,000	$2,000	$119,000
4%	25-year	$426,000	$190,000	$49,000	$16,000	$682,000
2%	50-year	$1,091,000	$434,000	$344,000	$63,000	$1,874,000
1%	100-year	$1,856,000	$682,000	$445,000	$116,000	$3,017,000
0.5%	200-year	$3,271,000	$1,169,000	$736,000	$181,000	$5,175,000
0.2%	500-year	$7,009,000	$2,785,000	$2,055,000	$266,000	$11,470,000
Breach		$2,078,000				
Average Annual Damages		$189,000				
Alternative 2 – Federal Decommission						
50%	2-year	$1,000	$1,000	$9,000	$0	$11,000
20%	5-year	$15,000	$8,000	$9,000	$2,000	$35,000
10%	10-year	$69,000	$27,000	$21,000	$2,000	$119,000
4%	25-year	$424,000	$189,000	$48,000	$16,000	$740,000
2%	50-year	$1,091,000	$434,000	$336,000	$63,000	$1,942,000
1%	100-year	$1,856,000	$682,000	$438,000	$116,000	$3,087,000
0.5%	200-year	$3,848,000	$1,405,000	$1,124,000	$199,000	$6,352,000
0.2%	500-year	$7,271,000	$2,913,000	$2,035,000	$274,000	$11,948,000
Average Annual Damages		$178,000				
Alternative 3 - HHPR						
50%	2-year	$1,000	$1,000	$9,000	$0	$11,000
20%	5-year	$15,000	$8,000	$9,000	$2,000	$35,000
10%	10-year	$69,000	$27,000	$22,000	$2,000	$119,000
4%	25-year	$426,000	$190,000	$49,000	$16,000	$682,000
2%	50-year	$1,091,000	$434,000	$344,000	$63,000	$1,874,000
1%	100-year	$1,856,000	$682,000	$445,000	$116,000	$3,017,000
0.5%	200-year	$3,271,000	$1,169,000	$733,000	$179,000	$5,169,000
0.2%	500-year	$6,826,000	$2,697,000	$1,950,000	$261,000	$11,147,000
Average Annual Damages		$188,000				
Alternative 4 – HHPR						
50%	2-year	$1,000	$1,000	$9,000	$0	$11,000
20%	5-year	$15,000	$8,000	$9,000	$2,000	$35,000
10%	10-year	$69,000	$27,000	$22,000	$2,000	$119,000
4%	25-year	$426,000	$190,000	$49,000	$16,000	$682,000
2%	50-year	$1,091,000	$434,000	$344,000	$63,000	$1,874,000
1%	100-year	$1,856,000	$682,000	$445,000	$116,000	$3,017,000
0.5%	200-year	$3,271,000	$1,169,000	$736,000	$179,000	$5,173,000
0.2%	500-year	$6,730,000	$2,670,000	$1,953,000	$261,000	$11,027,000
Average Annual Damages		$187,000				

The following summarizes the total annual average benefits quantified for each project alternative. To estimate the average annual damages associated with each alternative, the total damages were averaged between each recurrence interval and applied to the incremental probability between the respective flood events. The benefits for each FWFI alternative were estimated by comparing the damages that would occur under the alternative with those that would occur under the No Action alternative. The benefits were evaluated over the 100-year period of evaluation. A summary of total average annual damages avoided is provided in **Table 16.**

Table 16. Summary of Total Average Annual Damages Avoided (2024$)

Alternative	Total Average Annual Structural Damages	Total Average Annual Agriculture Damages	Total Average Annual Road Damages	Total Average Annual Damages	Total Average Annual Damages Avoided
Alternative 1 – No Action	$168,000	$3,000	$21,000	$192,000	$0
Alternative 2 – Federal Decommission	$172,000	$3,000	$23,000	$198,000	-$6,000
Alternative 3 – HHPR	$167,000	$3,000	$21,000	$191,000	$1,000
Alternative 4 – HHPR	$166,000	$3,000	$21,000	$190,000	$2,000

Note: all values rounded to the nearest thousand.

D.4 COST ANALYSIS

The average annual operation and maintenance (O&M) costs for each alternative were estimated. The net O&M costs for each FWFI alternative is the difference between the cost for the No Action alternative and the FWFI alternative (**Table 17**).

Table 17. Annual Operation and Maintenance Costs (2024$)

Alternative	Annual O&M Costs	Net Annual O&M Costs
Alternative 1 – No Action	$5,000	$0
Alternative 2 – Federal Decommission	$8,000	$3,000
Alternative 3 – HHPR	$5,000	$0
Alternative 4 – HHPR	$5,000	$0

Notes: 2024 price level, all values rounded to the nearest thousand.

The average annual costs associated with the alternatives and O&M costs of implementation for the No Action and FWFI alternatives are summarized in **Table 18**.

Table 18. Average Annual Costs of Alternative Implementation (2024$)

Alternative	Installation Costs	Average Annual Installation Costs	Net Annual O&M Costs	Average Annual Costs
Alternative 1 – No Action	$0	$0	$0	$0
Alternative 2 – Federal Decommission	$10,172,000	$312,000	$3,000	$315,000
Alternative 3 – HHPR	$10,764,000	$330,000	$0	$330,000
Alternative 4 – HHPR	$12,699,000	$390,000	$0	$390,000

Notes: 2024 price level; annualized over the 100-year evaluation period using a 2.75% discount rate; Average Annual Installation Cost includes interest during construction.

D.5 RESULTS OF THE ECONOMIC ANALYSIS

Benefits and costs over the period of analysis were annualized to allow for a direct comparison of average annual benefits to average annual costs. The benefits and costs used a price level of 2024 dollars and annualized using a discount rate of 2.75 percent over the 100-year evaluation period. **Table 19** summarizes the analysis results.

Table 19. Benefit-Cost Analysis Summary (2024$)

Alternative	Average Annual Costs	Average Annual Benefits	Average Annual Net Benefits	Benefit-Cost Ratio (BCR)
Alternative 1 – No Action	$0	$0	$0	1.0:1.0
Alternative 2 – Federal Decommission	$315,000	-$6,000	-$321,000	-0.0:1.0
Alternative 3 – HHPR	$330,000	$1,000	-$329,000	0.0:1.0
Alternative 4 – HHPR	$390,000	$2,000	-$388,000	0.0:1.0

Notes: 2024 price level; annualized over the 100-year evaluation period using a 2.75% discount rate; Average Annual Installation Cost includes interest during construction; all $ values rounded to the nearest thousand.

D.6 REGIONAL ECONOMIC ANALYSIS

A regional economic analysis was conducted by the NRCS economist. This calculated the regional impacts of the construction activities for the four alternatives, and the value-added flood damage reduction benefits using the IMPLAN model for the state of Texas. For the federally assisted alternatives (Alternative 3 and Alternative 4), most of the local cost-share dollars would be funded by a Texas State Government agency, not Karnes County, so it made more sense to use the state as the economic impacted area. The IMPLAN model was used, using standard NRCS procedures. The analysis was conducted for the recommended Alternative 3 & 4 as well as the decommissioning. **Table 20 to Table 24** below show the results of the regional economic analysis.

Table 20 Annual Flood Damage Benefits

IMPLAN Sectors	Benefits
6001 Proprietor Income	$0
10006 Households 70-100k	$23,571
Total	**$23,571**

Note: Proprietor Income – Farm Damages. Households – Structural and Infrastructure Damages

Table 21 Annual Flood Damage Impacts (Alt 3 & 4)

Annual Flood Damage Impacts	Impact Type	Employment	Labor Income	Value Added	Output
	Direct	-	$183,000.00	$183,000.00	$183,000.00
	Indirect	-	$0.00	$0.00	$0.00
	Induced	1.33	$78,753.38	$145,606.90	$255,753.36
	Total Effect	1.33	$261,753.38	$328,606.90	$438,753.36
Alternative 2 Damages		1.40	$275,930.63	$346,331.93	$462,324.54

Total Benefits Saved – Decommissioning vs Recommended Plan is $23,571 Annual Flood Benefits.

Table 22 Construction Costs

Cost Item	PL-83-566	Other funds	Total	IMPLAN Sectors	
Construction	$5,642,000	$3,038,000	$8,680,000	62	construction of highways, streets, bridges
Engineering	$868,000	$-	$868,000	457	Architectural, engineering, and related services
Permits		$174,000	$174,000	541	State Local Gov
Project Administration	$1,027,000	$15,000	$1,042,000	544	Federal Admin for Fed Share
Total	**$7,537,000**	**$3,227,000**	**$10,764,000**		

The construction costs are broken up and thus will have a different impact on the regional analysis.

Table 23 Construction Impacts

Impact Type	Employment	Labor Income	Value Added	Output
Direct Effect	49.15	$3,791,022.87	$5,245,806.33	$10,764,000.00
Indirect Effect	24.81	$1,894,074.11	$3,678,722.83	$7,124,377.63
Induced Effect	29.23	$1,702,960.61	$3,168,447.15	$5,580,434.68
Total Effect	103.19	7,388,057.59	12,092,976.31	23,468,812.31
Multipliers	9.59	0.69	1.12	2.18
	Jobs per $1m			

The construction impacts will lead to 10 jobs per $1 million spent. The total effect to the Texas Economy is nearly $23.5 million.

Table 24 Regional Economic Benefits

Regional Economic Benefits (Texas)	No Action	Alternative 2	Alternative 3 (Preferred Alt)	Alternative 4
Job-Years of Employment Created by Construction	$0	10.85	49.15	59.8
RED Direct Benefits to Texas Economy During Construction (One-time benefits)	$0	$2,879,423.08	$12,092,976.31	$14,869,720.57
Total RED Benefits During Construction to Texas Economy	$0	$ 5,986,516.27	$23,468,812.31	$28,890,528.03

D.7 APPENDIX A – STRUCTURE INVENTORY

D.7.1 Extent of Structure Inventory

An inventory was conducted for structures located within the inundation area associated with a 500-year flood event in the existing conditions.

D.7.2 Structure Inventory Cleanup

INITIAL PARCEL/STRUCTURE CLEANUP

Methodology:

1. Cleaned up the dataset for inundation so it was selectable.
2. Selected parcels that fell in the inundation zone to narrow the search.
3. Selected the buildings that fell on those parcels from step 2 to create a smaller dataset of buildings.
4. Selected the buildings from step 3 that intersected the inundation zone.
5. Re-selected the parcels that were associated with the buildings from step 4.
    a. Removed any parcels that did not fall in the use code list
6. Created centroids of the parcels from step 5 to create a “dot” of each impacted parcel.

Reducing selection set from above:

1. Clipped inundation layer to the impacted buildings.
2. Removed parcels with inundation less than 10 sq ft. impacting a structure.
3. Removed parcel records that had a “LevelNum” attribute of 2 or greater.

Spatial Join (Parcels):

1. Joined the parcel records from step 3 above and the buildings to get a count of structures per owner.
2. Created a field called “AECOM_Stru” to populate with the structure count by parcel. (This was done using the spatial join and intersect. Therefore, this resulted in multiple structures being listed as the structure(s) overlapped multiple parcels.)

Spatial Join (Buildings):

1. Created a spatial join between the building and the parcel(s) it intersects.
2. Created centroids of the buildings.
3. Jointed the building centroid with the dataset created in step 1.

STRUCTURE CLEANUP AND FLOOD DEPTHS

Methodology for Exhibit 2A structure points and “others” structure points:

1. The structure points were separated to be those within the area identified in Exhibit 2A and those that are not. They are identified as “others”.
2. To clean up the structure points in the Exhibit 2A area, aerial imagery was used, and the points were either moved to the primary structure or removed. This was done by using Esri default imagery and the point data sets.
3. Points were moved to the primary structures on a parcel. Points removed that were on sheds, garages, or other unoccupied structures.
4. None of the points for the “other” dataset were moved. They remained in their location and parcel as per the original structure dataset.