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Long Mott Energy, LLC

PSAR Subsection 2.4.3, “Probable Maximum Flood on Streams and Rivers”

CHAPTER 2

SUBSECTION 2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

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ACRONYMS AND ABBREVIATIONS

Acronym/Abbreviation	Definition
1D	one-dimensional
2D	two-dimensional
ac.	acre(s)
ACES	Automated Coastal Engineering System
ac-ft	acre-feet
ANSI	American National Standards Institute
ANS	American Nuclear Society
CEDAS	Coastal Engineering Design & Analysis System
CEM	Coastal Engineering Manual
cfs	cubic feet per second
CI	Conventional Island
cm	centimeter(s)
DWE	Diffusion-Wave Equation
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
fps	feet per second
ft.	feet
ft^2	square feet
ha	hectare(s)
HEC	Hydrologic Engineering Center

HEC-HMS	Hydrologic Engineering Centers Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Centers River Analysis System
HMR	NWS Hydrometeorological Report
hr.	hour(s)
HUC-12	12-Digit Hydrologic Unit Code
in.	inch(es)
km	kilometer(s)
km^2	square kilometer
kph	kilometers per hour
LMGS	Long Mott Generating Station
m	meter(s)
m^3	cubic meter
mi.	mile(s)
mi^2	square mile
mm	millimeter
mph	miles per hour
NAVD 88	North American Vertical Datum of 1988
NI	Nuclear Island
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NWS	U.S. National Weather Service

PMF	probable maximum flood
PMP	probable maximum precipitation
RFS	River Forecast System
s	second(s)
SCS	Soil Conservation Service
STA	Station
SWE	Shallow-Water Equations
TCEQ	Texas Commission on Environmental Quality
U.S.	United States
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
VCS	Victoria County Station
WGRFC	West Gulf Region Forecast Center
WRCM	Watershed Runoff Computer Model
WSEL	water surface elevation
yr.	year(s)

Chapter 2 Site Characteristics

2.4 HYDROLOGY

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

The Long Mott Generating Station (LMGS) site is located at the southern end of the lower Guadalupe River on the east bank of the river downstream of its confluence with the San Antonio River and just upstream of the United States (U.S.) Geological Survey (USGS) gage near Tivoli, Texas, as shown on Figure 2.4.3-1. The natural ground at the site varies in elevation from approximately 27 ft (8.2 m) to above 29 ft (8.8 m) in North American Vertical Datum of 1988 (NAVD 88). The finished floor grade of all safety-related structures is at elevation 31.5 ft (9.6 m) NAVD 88. Although the LMGS site is not located inside the Guadalupe River basin, flooding from this river is analyzed as there is a possibility of overtopping the bluff areas on the east side of the Guadalupe River and impacting the site.

Near the LMGS site, the Guadalupe River drains an area of about 5953 mi^2 (15,418 km^2). There are 29 dams upstream of the LMGS site on the Guadalupe River and its tributaries with storage capacity in excess of 3000 ac-ft (3.7 million m^3). The data pertinent to these dams, such as the type, dam height, top-of-dam elevation, storage volume, ownership, and location, are described in Section 2.4.1 and Section 2.4.4.

The most significant dams, in terms of flood storage capacity, are the Canyon Dam at river mile 303 on the Guadalupe River and the Coleto Creek Dam on Coleto Creek, a tributary of the Guadalupe River. The Canyon Dam has a top-of-dam elevation at 974.34 ft (296.98 m) NAVD 88 and a storage capacity of about 1.21 million ac-ft (1492.5 million m^3) at that level, as presented in the 2005 Probable Maximum Flood (PMF) Study Report for Canyon Dam by the U.S. Army Corps of Engineers (USACE) (USACE, 2005a). The Coleto Creek Dam has a top-of-dam elevation at 119.71 ft (36.49 m) NAVD 88 and a storage capacity of 149,800 ac-ft (184.8 million m^3), as given by U.S. National Weather Service (NWS) River Forecast System (RFS) for the Guadalupe River Basin down to Bloomington, Texas (NWS, 2007). See Section 2.4.3.1.

West Coloma Creek passes east of the LMGS site (see Figure 2.4.3-2). Because the safety-related facilities are located inside of its watershed, flooding of this creek due to probable maximum precipitation (PMP) is considered and analyzed separate from Guadalupe River.

Flooding ~~from~~ analysis for the San Antonio River is presented in Subsection 2.4.3.3. ~~not considered as it has a smaller watershed area (i.e., 4194 mi^2 [10,862 km^2]) than the Guadalupe River~~

~~and a~~A simultaneous PMP event on both watersheds is not reasonable. Instead, according to American National Standards Institute/American Nuclear Society (ANSI/ANS) document ANSI/ANS 2.8-1992, proper river discharge is considered simultaneous with PMP over Guadalupe River watershed as described in following subsections (ANSI/ANS, 1992).

Based on air temperature data given in Subsection 2.4.7, the low probability occurrence of snow within the Lower Guadalupe River Basin and its effect on flood-producing phenomena indicated that snow-melt and antecedent snow-pack are not critical factors in the production of floods at the LMGS site. In addition, because the drainage area of the Guadalupe River at the LMGS site is about 5953 mi^2 (15,418 km^2), it is not expected that any urban developments in the basin would significantly alter the flood runoff characteristics of the watershed or the flood level at the LMGS site. Regarding the West Coloma Creek watershed, conservative assumptions are made to address such uncertainty.

In this subsection, the effects of the PMF in the Lower Guadalupe River and PMF in the West Coloma Creek on the safety-related facilities of LMGS site are evaluated. The effects of flooding resulting from local intense precipitation and from potential dam failures are addressed, respectively, in Section 2.4.2 and Section 2.4.4.

The following major hydrologic and hydraulic studies on the Guadalupe River Basin were performed by federal, state, and other local agencies. These studies/analyses include:

- USACE, Reconnaissance Report, Canyon Lake Modification of Embankment, Guadalupe River, Texas, Fort Worth District, October 1979 (USACE, 1979).
- USACE, Dam Assurance Study on Canyon Lake, Guadalupe Basin, Texas, Fort Worth District, June 2005 (USACE, 2005a).
- USACE, Flood Forecast Model for Guadalupe River Basin, HEC-1 input data file, updated on August 12, 2004 (USACE,2004) with data on the Soil Conservation Service (SCS) reservoirs updated on February 4, 2008.
- Albert H. Halff Associates, Inc., Dam Break Analysis for Coleto Creek Dam, for Guadalupe-Blanco River Authority, March 1989 (Halff, 1989).
- Albert H. Halff Associates, Inc., Phase 1 Hydrologic Study, Coleto Creek Dam, prepared for the Guadalupe-Blanco River Authority, December 1992 (Halff, 1992).
- URS Corporation, Bi-Annual Dam Inspection of Coleto Creek Dam, submitted to Guadalupe-Blanco River Authority, August 21, 2003 (URS, 2003).
- Federal Emergency Management Agency (FEMA), Flood Insurance Study (FIS), Victoria County, Texas, Unincorporated Area, November 20, 1998 (FEMA, 1998).
- FEMA, FIS, City of Victoria, Texas, Victoria County, July 21, 1999 (FEMA, 1999).
- InFRM, Watershed Hydrology Assessment for the Guadalupe River Basin, September 2019 (InFRM, 2019).
- Victoria County Station (VCS) nuclear plant Early Site Permit Application (VCS, 2007).

Among the existing flood study reports and data files listed above, there are four PMF studies performed for the Guadalupe River Basin. The first study was conducted by the USACE, who studied the Canyon Dam in the Upper Guadalupe River for a drainage area of about 1425 mi^2 (3691 km^2) (USACE, 2005a and USACE, 1979). The second study was performed by Albert H. Halff Associates, Inc., for the Coleto Creek Dam on Coleto Creek, with a drainage area of about 491 mi^2 (1272 km^2), for the Guadalupe-Blanco River Authority (Halff, 1992). The third study is the InFRM analysis performed in 2019 that covers a large portion of the Guadalupe River Basin, but not the entire area. The main goal of the InFRM analysis was to establish the 100-yr. flood event, but it also provides 500-yr. flood event results. The fourth analysis is the one performed as part of the Early Site Permit application to the U.S. Nuclear Regulatory Commission (NRC) for the ~~Victoria County Station~~VCS in 2007.

~~Because these studies are not directly applicable to the LMGS site, an independent PMF model study is being performed to evaluate the flooding impact on the safety-related facilities of LMGS. The results of the VCS, 2007 analysis are provided in this submittal. Additional site-specific analyses and associated information that includes the postulated coincidental wind setup and wave setup will be provided by the end of 2025.~~

The PMF study performed for VCS adopted the basin characteristics from the USACE Flood Forecast Model (USACE, 2004), including the physical layout of the watershed elements such as sub-basin boundary definition, channel reach locations and physical characteristics, and dam/reservoir physical attributes.

In addition to the VCS analysis, an independent PMF analysis on the Guadalupe and San Antonio watersheds was performed as documented in Subsection 2.4.3.3

No study was found for the flooding in West Coloma Creek and therefore an independent analysis is performed as documented in Subsection 2.4.3.2.

2.4.3.1 Summary of VCS PMF Analysis

The PMF hydrographs due to PMP over Guadalupe River Watershed were developed using the computer application HEC-HMS (Hydrologic Engineering Centers Hydrologic Modeling System) (USACE, 2006b). The USACE Flood Forecast Model, which was developed in HEC-1 (USACE, 1990), was first converted to HEC-HMS format. The HEC-HMS model was then expanded to include the Coleto Creek Watershed using model data from the NWS RFS for the Guadalupe River Basin (NWS, 2007) and Halff Associates' model study for the Coleto Creek Watershed (Halff, 1992), as well as records from the USGS stream gaging stations.

The VCS analysis was based on USACE, 2005a and the calibrated basin runoff model used for the PMF development of the Canyon Dam Watershed was, therefore, applied directly in the VCS PMF model. The revised expanded model was further calibrated for the portion of the Guadalupe River Basin downstream of the Canyon Dam only.

The HEC-HMS model for the portion of the Guadalupe River Basin downstream of the Canyon Dam was calibrated during the VCS study using the observed rainfall and flood hydrograph data from two storms, which occurred in October 1998 and November 2004. According to the flood peak discharge data observed at the USGS gage on the Guadalupe River at Victoria, Texas, these are still the largest and fourth largest floods on record from 1935 to 2023. The fourth largest flood on record, the 2004 flood, was selected because the average basin rainfall data for the Guadalupe River Basin for the second largest flood (July 1935) and third largest flood (September 1981) on record are not available.

The calibrated HEC-HMS model for the Guadalupe River Basin downstream of the Canyon Dam to the LMGS site was expanded to include the Canyon Dam Watershed using the basin model parameters of the 2005 USACE PMF Study for that watershed (USACE, 2005a). A verification study was performed to ensure that the calibrated Watershed Runoff Computer

Model (WRCM) HEC-1 model for the Canyon Dam Watershed was properly converted to the HEC-HMS model.

The 72-hr PMP estimates for the Guadalupe River near the LMGS site are derived following procedures described in NWS Hydrometeorological Reports (HMRs) Nos. 51 and 52 (NWS, 1978 and NWS, 1992 respectively), and using HEC-HMR 52, developed by Hydrologic Engineering Center (HEC) of the USACE (USACE, 1987).

For the watershed downstream of the Canyon Dam, the precipitation losses for the PMF model were conservatively established from the model calibrations of the 1998 and 2004 floods. For the Canyon Dam watershed, the precipitation losses used by the USACE in the 2005 PMF Study (USACE, 2005a) were adopted as given.

The Guadalupe River PMF hydrograph at the LMGS site was found to have a peak discharge of about 1,300,000 cfs (36,812 m^3/s). The maximum PMF still water level at the LMGS site is estimated to be at elevation 30.7 ft (9.37 m) NAVD 88. This elevation is the average of Station (STA) 8.7744 and STA 11.1811 from HEC-RAS (Hydrologic Engineering Centers River Analysis System). The left bank elevation at this location is ~~33.91~~41.72 ft (12.7~~10.34~~ m) NAVD 88, which indicates that the left bank of the river is not overtopped at the site location.

A 500-yr. flood event in the San Antonio River is postulated to be occurring coincidentally with the PMF event in the Guadalupe River. The 500-yr. flood flow from the San Antonio River is estimated to be about 180,000 cfs (5097 m^3/s).

2.4.3.1.1 Probable Maximum Precipitation (PMP)

PMP depths for the Guadalupe River Basin are derived following the procedures described in NWS HMR Nos. 51 and 52 (NWS, 1978 and NWS, 1992 respectively) and using the computer program HEC-HMR 52 (USACE, 1987).

In using HEC-HMR 52, the PMP estimates and the storm orientation for the basin of interest for the various area sizes and durations are required as inputs to the program. They are derived from NWS HMR Nos. 51 and 52 and are presented in Table 2.4.3-1.

HEC-HMR 52 also requires the X and Y coordinates of the boundaries of the river basin and of each of the sub-basins, as well as the preferred storm orientation, which is 195 degrees~~°~~, as suggested in NWS HMR No. 52 for this area. The boundaries of the Guadalupe River Basin and its sub-basins are shown on Figure 2.4.3-3. The program estimates the hourly PMP values for each of the sub-basins for a particular storm center in the basin and the hourly PMP values are stored in the data storage system (USACE, 2006a) to be recalled for use in flood hydrograph developments.

In accordance with guidelines suggested by ANSI/ANS 2.8-1992, Subsection 9.2.1.1 (ANSI/ANS, 1992), an antecedent storm, equal to 40 percent of the 72-hr PMP, is assumed to end three days before the start of the 72-hr PMP.

2.4.3.1.2 Precipitation Losses

For the watershed downstream of Canyon Dam, the adopted initial losses for the sub-basins vary from 0.05 in. (1.27 mm) to 1.0 in. (25.4 mm), while the constant loss rates of 0.05 in/hr

(1.27 mm/hr) and 0.1 in/hr (2.54 mm/hr) are used. These losses are derived primarily from the results of the model calibrations of the 1998 and 2004 floods and adjusted conservatively for use in the PMF developments. For the Canyon Dam Watershed, the values used by the USACE in its 2005 PMF study, an initial loss of 1.0 in. (25.4 mm) and a constant loss of 0.15 in/hr (3.81 mm/hr), with the losses for the surface of Canyon Lake being zero, are adopted. The loss values for each of the sub-basins used in the PMF development are presented in Table 2.4.3-2.

2.4.3.1.3 Runoff and Stream Course Models

The PMF model adopts the basin characteristics from the Flood Forecast Model developed by the USACE, Fort Worth District, Texas, for the Guadalupe River Basin down to the USGS gage at Victoria, Texas (USACE, 2004). The basin characteristics include the physical layout of the watershed elements, such as sub-basin boundary definitions, channel reach locations and physical characteristics, and dam/reservoir physical attributes for basin runoff calibration. The USACE model includes the Canyon Dam because of its large flood storage capacity. In addition, two small agriculture-related reservoirs located in the San Marcos River Basin were modeled. They are the SCS No. 3 and SCS No. 5 reservoirs with the top of dam elevations at 648.5 ft (197.66 m) NAVD 88 and 667.2 ft (203.36 m) NAVD 88, and maximum storage capacities of about 4000 ac-ft (4.93 million m^3) and 7000 ac-ft (8.63 million m^3), respectively (USACE, 2004).

The USACE model only covers the portion of the basin upstream of the USGS gage at Victoria, Texas. It does not include the drainage area from Coleto Creek, a tributary of Guadalupe River, which joins the main river downstream of the gage at Victoria. The Coleto Creek Watershed, together with the Coleto Creek Dam/Reservoir, is modeled by including the drainage areas given by the USGS at gaging station No. 08176900 (USGS, 2008) and by Halff Associates for the Coleto Creek Dam/Reservoir (Halff, 1992). The drainage area downstream of the Coleto Dam to its confluence with Guadalupe River, the sub-basin boundaries, and the elevation-storage-discharge relationships for the Coleto Creek Dam/reservoir are those given in the NWS RFS for the Guadalupe River Basin near Bloomington, Texas (Station DUPT2) (NWS, 2007).

In the 1979 PMF study for the Canyon Dam (USACE, 1979), USACE calibrated the runoff response characteristics of the watershed with the August 1978 flood flows observed at the Johnson Creek gage near Ingram, the North Fork gage near Hunt, and the Guadalupe River gages near Hunt, Comfort, and Spring Branch. USACE updated the model in 2005 (USACE, 2005a) using the WRCM (USACE, 1985) with the same basin runoff response characteristics, but including additional PMP data from NWS HMR 52 (NWS, 1992). The 1978 flood is still the flood of record to date for the part of the Guadalupe Watershed upstream of Canyon Dam, which has a drainage area of about 1432 mi^2 (3709 km^2) (Table 2.4.3-3 and Table 2.4.3-4). It is, therefore, reasonable to postulate that the runoff parameters established in the USACE PMF model adequately represent the basin response during extreme floods and no new calibration of the Canyon Dam Watershed is necessary. The calibration efforts, therefore, concentrate only on the portion of the Guadalupe Watershed downstream of the Canyon Dam.

The resulting composite watershed and the sub-basins, including those for the Canyon Dam Watershed used in the 2005 USACE PMF model, are shown on Figure 2.4.3-3.

2.4.3.1.4 Runoff Model Calibrations

The HEC-HMS model developed for the watershed downstream of Canyon Dam is calibrated using the flood records of the October 1998 and November 2004 storms. The storms of October 1998 and November 2004 produced the largest and the fourth largest floods on record from 1935 to 2006 at the USGS gaging station at Victoria, Texas, as indicated in Table 2.4.3-5. The observed hourly rainfall depths for each of the sub-basins, from

October 13, 1998, to October 30, 1998, and from November 15, 2004, to December 3, 2004, are obtained from the NWS West Gulf Region Forecast Center (WGRFC). There are a total of 13 USGS stream gaging stations on the Guadalupe River downstream of the Canyon Dam to the LMGS site for which flood hydrograph data are available for the 1998 and 2004 floods. The observed 15-minute flood flow hydrographs from these gages are used for the calibration (USGS, 2008). The gaging stations are:

- USGS No. 08168500 - Guadalupe River above Comal River with a drainage area of 1518 mi^2 (3932 km^2)
- USGS No. 08173900 - Guadalupe River at Gonzales with a drainage area of 3490 mi^2 (9039 km^2)
- USGS No. 08175800 - Guadalupe River at Cuero with a drainage area of 4934 mi^2 (12,779 km^2)
- USGS No. 08176500 - Guadalupe River at Victoria with a drainage area of 5198 mi^2 (13,463 km^2)
- USGS No. 08171000 - Blanco River at Wimberley with a drainage area of 355 mi^2 (919 km^2)
- USGS No. 08171300 - Blanco River near Kyle with a drainage area of 412 mi^2 (1067 km^2)
- USGS No. 08172000 - San Marcos River at Luling with a drainage area of 838 mi^2 (2170 km^2)
- USGS No. 08172400 - Plum Creek at Lockhart with a drainage area of 112 mi^2 (290 km^2)
- USGS No. 08173000 - Plum Creek near Luling with a drainage area of 309 mi^2 (800 km^2)
- USGS No. 08174600 - Peach Creek below Dilworth with a drainage area of 460 mi^2 (1191 km^2) (2004 Storm Only)
- USGS No. 08175000 - Sandies Creek near Westhoff with a drainage area of 549 mi^2 (1422 km^2)

- USGS No. 08176900 - Coleto Creek at Arnold Road Crossing near Schroeder with a drainage area of 357 mi^2 (925 km^2)
- USGS No. 08177500 - Coleto Creek near Victoria with a drainage area of 514 mi^2 (1331 km^2)

In the calibration of the HEC-HMS model, the 1998 and 2004 observed flood hydrographs of Guadalupe River at the Sattler, Texas, gage (USGS Gage No. 08167800) are used as inflows to the model basin. The Sattler gage is located immediately downstream of Canyon Dam with a drainage area of 1436 mi^2 (3719 km^2). The locations of the USGS gauging stations are shown in Figure 2.4.3-1.

The model basin of the USACE Flood Forecast Model is subdivided into 22 sub-basins, linked respectively by 16 channel reaches. The model uses the Muskingum channel routing method for 13 of these 16 channels reaches and the Modified Puls method with prescribed storage-discharge relationship for the remaining three channel reaches. In the calibration process, only the K and X values are adjusted and the storage-discharge relationships remain unchanged as defined by the USACE.

As noted in Section 2.4.3.1, the USACE Flood Forecast Model does not include the Coleto Creek Watershed. Thus, the Coleto Creek Watershed and Reservoir are added to the model. The stage-storage and storage-discharge relationships for Coleto Creek Dam/Reservoir from NWS RFS for the Guadalupe River Basin are adopted instead of those from URS, 2003 because they are more current (dated March 2007) and are used by NWS WGRFC in its current flood forecast model for the Guadalupe River Basin. The runoff model of the Coleto Creek Watershed at its confluence with the Guadalupe River is represented by three sub-basins, Sub-basins 29, 30, and 31 (see Figure 2.4.3-3), and three channel reaches.

The historical observed rainfall data for the 1998 and 2004 floods are obtained from NWS WGRFC for the sub-basins shown on Figure 2.4.3-4. Comparisons of the individual drainage boundaries of the respective sub-basins, as shown on Figure 2.4.3-3 and Figure 2.4.3-4, indicate that there are some minor differences in the sub-basin definitions. In some parts of the basin, the USACE definitions are more refined, resulting in more sub-basins, and the reverse is true for other areas. In areas where a USACE sub-basin consists of more than one NWS sub-basin, the area-weighted average of the NWS sub-basin rainfall depths is used to approximate the rainfall depth of the corresponding USACE sub-basin. In the case where the NWS sub-basin encompasses a number of USACE sub-basins, the average rainfall depth of that NWS sub-basin is assumed to be applicable to all the corresponding USACE sub-basins. Table 2.4.3-6 provides the names of the NWS sub-basin rainfall data files used as inputs to the HEC-HMS for the sub-basins downstream of Canyon Dam. With a given drainage area and rainfall input sequence, the runoff characteristics of a basin are defined by four groups of parameters in the HEC-HMS rainfall-runoff model, namely:

- Basin losses
- Runoff characteristic of rainfall excess to the conveyance channels
- Base flow characteristics
- Channel and reservoir flood routing characteristics

For the VCS PMF study, the basin losses are represented by an initial loss and a constant loss rate. The Snyder's lag and peaking are used to define the runoff characteristics. Further calibration is provided in Subsection 2.4.3 of VCS, 2007.

The plots of the observed and computed flood hydrographs at each of the USGS gaging stations used in the calibration process for the 1998 and 2004 floods are depicted in Figure 2.4.3-5 through Figure 2.4.3-25. The calibrated basin runoff parameters, namely, the basin loss values, the Snyder's lag and peaking coefficients, the base flow recession coefficients, and Muskingum K and X values for the 1998 and 2004 floods, are also presented in Table 2.4.3-7 through Table 2.4.3-10.

2.4.3.1.5 Probable Maximum Flood Flow

The adopted basin runoff parameters for the PMF development for each of the basin elements for the Guadalupe River Basin downstream of the Canyon Dam are shown in Table 2.4.3-2 and Table 2.4.3-11. They are developed from the calibrated basin runoff parameters given in Table 2.4.3-7 through Table 2.4.3-10 by selecting the more conservative values of the two. To account for nonlinearity effects of extreme flood conditions, the calibrated Snyder's lags for the sub-basins are reduced by 15 to 20 percent. A lag reduction is typically suggested for PMF development (USACE, 1994) even though the model is calibrated with the extreme flood of October 1998.

In the 2005 PMF study for the Canyon Dam (USACE, 2005a), the USACE subdivided the watershed above the dam into nine sub-basins instead of the 24 in the Flood Forecast Model (USACE, 1979). The definitions of the sub-basins and respective drainage areas in the 2005 study are presented in Table 2.4.3-12. Note that the total drainage area for the Canyon Dam is shown as 1417.85 mi^2 (3672 km^2), which is slightly less than the drainage area of 1436 mi^2 (3719 km^2) given by USGS. The sub-basin delineation for the Flood Forecast Model is modified for the Canyon Dam Watershed to match those given in the 2005 PMF study for the same area, as shown in Figure 2.4.3-3. These revised sub-basin boundaries, including the watershed for the Coleto Creek Basin, are used in the PMF model development.

In the same 2005 study, the unit hydrographs for each of the sub-basins were specified by the USACE and are presented in Table 2.4.3-13. A storage-discharge relationship was defined by the USACE for each of the five channel elements for flood routing through the channel reaches in the watershed, as shown in Table 2.4.3-14. The elevation-storage-discharge relationship for the Canyon Dam and reservoir is presented in Table 2.4.3-15. The basin runoff routing parameters for each of these sub-basins are obtained from the WRCM.

The hourly PMP estimates for each of the sub-basins, with storm centers as given in Table 2.4.3-16, are used as input to the calibrated PMF HEC-HMS model with the loss and base flow parameters as given in Table 2.4.3-2.

In accordance with the combined-event criterion stated in Subsection 9.2.1.1 of ANSI/ANS 2.8-1992 (ANSI/ANS, 1992), an antecedent rainfall equal to 40 percent of the PMP should be simulated as part of the PMF flood level determination. As suggested in Subsection 5.2.7.1 of ANSI/ANS 2.8-1992 (ANSI/ANS, 1992), an antecedent storm preceding the PMP by three days is selected. Using a 72-hr PMP, this combined-event criterion requires generating a 40 percent PMP sequence and placing it six days ahead of the PMP estimates. To simulate this, the HEC-HMS is first run using the 40 percent PMP event. The simulation is re-started for the full PMP event with the starting water levels in the four reservoirs, Canyon Lake, SCS Nos. 3 and 5, and the Coleto Creek Reservoir, equal to the predicted reservoir levels at three days after the cessation of the 40 percent PMP event, as shown in Table 2.4.3-17.

2.4.3.1.6 Water Level Determinations

The water surface profile in the Guadalupe River for the postulated PMF condition is estimated using the steady state routing option of the computer program HEC-RAS, version 3.1.3 (USACE, 2005b). The river channel and cross section geometry of the Guadalupe River from San Antonio Bay are established from the digital terrain map of the area (USGS, 2007). The Manning's n values are conservatively assumed to be 0.1 for both the channel and over-bank areas. Section 2.4.4 discusses the selection of this Manning's n value. The locations of these cross sections are presented in Figure 2.4.3-26. The downstream boundary condition is assumed to be at normal depth with a slope equal to 0.00016, which is the average of the bed slopes between the 10-ft to 20-ft (3- to 6- m) contour, equal to 0.00032, and that near the confluence between Guadalupe and San Antonio Rivers, which is basically flat.

The San Antonio River joins the Guadalupe River upstream of Tivoli, Texas. For the PMF prediction, a 500-yr. flood in the San Antonio River Basin is assumed to occur coincidentally with a PMF event in the Guadalupe River Basin.

The USGS gaging station on the San Antonio River closest to its confluence with the Guadalupe River and with a long stream flow record for flood frequency analysis is at Goliad, Texas (USGS Gage No. 0818850). At this gage, the San Antonio River drains an area of about 3921 mi^2 (10,155 km^2). A flood frequency analysis is performed using 75 yr. of data, assuming the Log-Pearson Type III distribution and following the formulations suggested by Hame~~a~~d and Rao (Hamad and Rao, 2000) and USGS Bulletin 17 B (USGS, 1982). The 500-yr. flood peak discharge at Goliad is found to be about 164,000 cfs (4644 m^3/s).

The San Antonio River drainage area at its confluence with the Guadalupe River is estimated to be about 4180 mi^2 (10,826 km^2). By prorating the peak discharge using a drainage area ratio, the San Antonio River 500-yr. flood peak discharge at its confluence with the Guadalupe River is determined to be 180,000 cfs (5097 m^3/s). This flow rate is added to the Guadalupe River PMF peak discharge of 1,123,300 cfs (31,808.3 m^3/s), yielding a total flood discharge of about 1,303,300 cfs (36,905 m^3/s).

The PMF peak discharge value of 1,300,000 cfs (36,812 m^3/s) is used for the cross sections downstream of the confluence of the San Antonio River in the HEC-RAS model in determining the PMF water level. Upstream of that confluence, the PMF peak discharge used in the model is 1,120,000 cfs (31,715 m^3/s). The PMF inflow discharge from Coleto Creek to the Guadalupe River at the time of the PMF peak discharge in the Guadalupe River is estimated to be about 20,000 cfs (566 m^3/s) from the HEC-HMS run. Therefore, for the reach of the Guadalupe River upstream of its confluence with Coleto Creek, the PMF peak discharge used in the simulation is about 1,100,000 cfs (31,149 m^3/s). The PMF water surface profile along the Guadalupe River from STA 7.247 to STA 66.256 is shown in Table 2.4.3-18. The PMF flooding water level of the Guadalupe River near the LMGS site (average of river mile 11.1811 and 8.7744) is found to be about 30.74 ft (9.37 m) NAVD 88.

The HEC-RAS model does not include the inflatable Lower Guadalupe Salt Water Barrier and Diversion Dam (Fabridam) located near river mile 11. Because this inflatable dam would rupture when a hydraulic head against the dam exceeds about 4.8 ft (1.46 m), when inflated (GBRA, 1994), it would not have any effect on the PMF level.

~~2.4.3.1.7 Coincident Wind-Wave Activity~~

~~The 2-yr. mean recurrence interval annual extreme-mile wind speed was obtained from ANSI/ANS 2.8-1992 (ANSI/ANS, 1992). Wave runup is estimated to be 6.4 ft (1.95 m) for a 50 mph (80.46 kph) design wind speed.~~

~~Wind setup is estimated to be 1.8 ft (0.55 m).~~

~~2.4.3.1.8~~2.4.3.1.7 Probable Maximum Flood Water Level over Guadalupe River

The maximum PMF still water level of the Guadalupe River at the LMGS site, before wind-wave induced setup and run-up, is predicted to be at elevation 30.74 ft (9.37 m) NAVD 88. ~~Adding the conservative combined wind setup and maximum wave run-up prediction of about 8.2 ft (2.5 m), the maximum PMF flooding water level at the LMGS site is postulated to be at elevation 38.94 ft (11.87 m) NAVD 88. This is higher than the 33.91 ft (10.34 m) NAVD 88 by 5.03 ft (1.53 m). Therefore, the left bank of the river will be overtopped by wind-wave runup effect. The site will not be flooded during Guadalupe River PMF event as the overtopped spillage will be distributed over the low-lying area behind the riverbank toward the site. Additionally, the site is raised by a minimum of 4 ft (1.2 m) from the low-lying areas surrounding the site.~~

2.4.3.2 West Coloma Creek Probable Maximum Flood

2.4.3.2.1 Watershed Description

West Coloma Creek watershed is in Calhoun County, Texas. To identify West Coloma Creek watershed, the 12-Digit Hydrologic Unit Code (HUC-12) watershed boundaries were used. The adjacent drainage boundaries with their respective names, along with the general topography of the area, are shown in Figure 2.4.3-27. The HUC-12 boundaries near the site were refined and a direct tributary area was delineated and presented in Figure 2.4.3-28. The entire watershed runs from downstream to upstream from Powderhorn Lake in Matagorda Bay to approximately 4.5 mi (7.2 km) north of Green Lake (see Figure 2.4.3-2). West Coloma Creek watershed has a size of 102.32 mi^2 (265.01 km^2). The drainage area north of Farik Road is 9.1 mi^2 (1.7 mi^2 + 5.5 mi^2 + 1.9 mi^2) (4.4 km^2+ 14.2 km^2 + 4.9 km^2) (23.5 km^2). The drainage area south of Farik Rd. upstream of Jesse Rigby Road (i.e., the LMGS site) is 4.6 mi^2 (11.9 km^2). Therefore, the total watershed area upstream of the LMGS site is 23 mi^2 (60 km^2) (Figure 2.4.3-29). Of this area, 15.5 mi^2 (40.1 km^2) flows toward the site and the rest is part of East Coloma Creek. The portion of the watershed that is upstream of Sparks Road is 7.2 mi^2 (18.6 km^2) and is called Sparks Watershed in this analysis (Figure 2.4.3-30).

The 2018 USGS LiDAR data for South Texas (USGS, 2018) with nominal pulse spacing of 2.3 ft (0.8 m) and vertical accuracy of 8 in. (5.8 cm) at a 95 percent confidence level is used in this analysis. The data were developed based on a horizontal projection/datum of North American Datum of 1983 (2011), Universal Transverse Mercator Zone 14, meters and vertical datum of NAVD 88 (GEOID12B) meters. Elevations on the north and south side of Farik Road are presented in Figure 2.4.3-31 and Figure 2.4.3-32. They depict the approximate overtopping elevations that would need to be achieved for water to flow from the north tributaries into the site's direct tributary. Topography of the watershed downstream of the Sparks Road is presented in Figure 2.4.3-33 and shows that the watershed elevations vary from 3 ft (0.9 m) NAVD 88 to 39 ft (12 m) NAVD 88 from downstream to upstream with the existing grade varying from at 26 ft (7.9 m) NAVD 88 to 28 ft (8.5 m) NAVD 88 around the LMGS site.

2.4.3.2.2 Probable Maximum Precipitation ~~(PMP)~~

PMP depths for the West Coloma Creek basin are derived following the procedures described in NWS HMR Nos. 51 and 52 (NWS, 1978 and NWS, 1992 respectively).

In using HEC-HMR 52, the PMP estimates and the storm orientation for the basin of interest for the various area sizes and durations are required as inputs to the program. They are derived from NWS HMR Nos. 51 and 52 and are presented in Table 2.4.3-19. Instead of considering multiple storm centers and developing multiple PMPs, conservatively, the 10-mi^2 (26-km^2) PMP developed for the site is applied to the entire watershed. This is reasonable because the direct contributing watershed upstream of the LMGS site is only 15.5 mi^2 (40.1 km^2). 6-hourly precipitation depths were developed by interpolating between the 12- and 24-hr, 24- and 48-hr, and 48- and 72-hr depths (see Table 2.4.3-20).

In accordance with guidelines suggested by ANSI/ANS 2.8-1992, Subsection 9.2.1.1 (ANSI/ANS, 1992), an antecedent storm, equal to 40 percent of the 72-hr PMP, is assumed to end 3 days before the start of the 72-hr PMP.

The standardized distribution for a 72-hr PMP storm event from HMR 52, Section 3.1, page 16, is used to establish the PMP hyetograph. Table 2.4.3-21 presents the order of incremental PMP rainfall. The entire applied PMP hyetograph including the antecedent rainfall is presented in Figure 2.4.3-34.

2.4.3.2.3 Precipitation Losses

The SCS curve number method is a standard, widely used, and efficient method for determining the amount of runoff from rainfall events. A combination of land use and hydrologic soil group are used to determine curve number. The Conterminous U.S. land cover (i.e., the National Land Cover Database [NLCD]) at a 30-m (98-ft) spatial resolution with a 16-class legend based on a modified Anderson Level II classification system (USGS, 2021) is used and presented in Figure 2.4.3-35. The majority of the basin is covered with cultivated crops or hay/pasture.

Soils are classified by the Natural Resource Conservation Service into four Hydrologic Soil Groups, A, B, C, and D, based on the soil's runoff potential, where soils classified as A generally have the smallest runoff potential and soils classified as D the greatest. The Soil Survey Geographic Database is used in this analysis (NRCS, 2024) and presented in Figure 2.4.3-36. Most of the basin is covered with soil type D.

Table 2.4.3-22 shows the curve number for each soil group for each land use category. As described above, for the combination of hydrologic soil type D and agricultural land use in the watershed, the corresponding curve number is 87. A curve number of 90 is conservatively applied to the entire watershed.

2.4.3.2.4 Runoff Hydrograph Determination

The Sparks watershed presented in Figure 2.4.3-30 combined with the Farik watershed contribute flow to the site. Sensitivity analysis showed that Farik Road is overtopped during a PMP event. The majority of the runoff hydrograph is captured by rain on grid in HEC-RAS 2D model Version 6.4.1 (See Section 2.4.3.2.5). However, the HEC-HMS Version 4.11 hydrology model is used to estimate the runoff hydrograph of the Sparks watershed as the upstream boundary condition, which is outside the footprint of the HEC-RAS 2D model, presented in Figure 2.4.3-37.

The following conservative inputs were applied to the HEC-HMS model:

- Curve number of 90 (see Section 2.4.3.2.3)
- Lag time is calculated as 5 hr, but, conservatively, a lag time of 10 minutes~~.~~ was applied
- PMP hyetograph from Section 2.4.3.2.2 (Figure 2.4.3-34)
- Although impervious area is less than 5 percent, conservatively, 20 percent impervious area was used in the model

The runoff hydrograph generated for the Sparks watershed as the upstream boundary condition is presented in Figure 2.4.3-38 with maximum flow associated with 40 percent PMP and full PMP estimated as 10,000 cfs (283 m^3/s) and 25,000 cfs (708 m^3/s), respectively.

2.4.3.2.5 Hydraulic Model Setup and Flood Routing

A detailed, two-dimensional (2D) flood routing model developed for the LMGS site and its direct watershed (see Figure 2.4.3-37) is used to establish depth of flooding and maximum velocities. This model, HEC-RAS 2D (USACE, 2020), represents all the topographical and man-made features (i.e., buildings, tanks, and hydraulic structures) that significantly affect runoff at the LMGS site and its local watershed.

By using a 2D model, floodwater is routed in a natural manner without being “forced” to flow in predefined directions. This allows for a more accurate flood analysis than is possible with one-dimensional (1D) models. The HEC-RAS 2D Reference Manual (USACE, 2020) describes the HEC-RAS model as follows: “HEC-RAS is designed to perform one-dimensional (1D), two-dimensional (2D), or combined 1D and 2D hydraulic calculations for a full network of natural and constructed channels. HEC-RAS solves the Diffusion-Wave Equation (DWE) and Shallow-Water Equations (SWE).” Additionally, the HEC-RAS model is approved by FEMA FIS (FEMA, 2020). HEC-RAS is also approved by the NRC to perform local intense precipitation analysis, as described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America.

Topographic LiDAR (USGS, 2018) was used for construction of a terrain model used for the topography and bathymetry of the model domain. The extent of the HEC-RAS 2D model is illustrated in Figure 2.4.3-37.

The model boundaries are established away from the Nuclear Island/Conventional Island (NI/CI) area and safety-related structures, systems, and components to prevent boundary conditions from affecting flood levels evaluated within the NI/CI area and to ensure the stability of the model. The two ditches upstream of Farik Road redirect part of the upstream runoff toward Green Lake (see Figure 2.4.3-30). Farik Road is overtopped during a PMP event and HEC-RAS 2D determines the flow that passes over the road and flows toward the site. Model boundary conditions are comprised of the following:

- Upstream: Sparks basin flow hydrograph (Figure 2.4.3-38)
- Downstream: Normal flow boundary condition with conservative slope of 0.001
- Western Boundary: selected based on sensitivity analysis and set to normal flow with slope of 0.001
- Entire model domain: Rain on grid with hyetograph presented in Figure 2.4.3-34

The HEC-RAS 2D model covers an area of approximately 47,836 ac. (19,359 ha); about 5.8 mi (9.3 km) in the east-west direction and about 20.7 mi (33.3 km) in the northwest-southeast direction (Figure 2.4.3-37). The numerical grid was generated with the RAS Mapper module. The horizontal grid size was 200 by 200 ft (61 by 61 m) within the entire model domain. The model grid is refined to 10 by 10 ft (3 by 3 m) at West Coloma Creek, the edges of the embankments, and ditches around the site. The model has 275,235 cells (Figure 2.4.3-39). Sensitivity analysis on the size of the mesh was performed and confirmed that the selected cell size results in similar output as smaller cells (i.e., 50 ft x 50 ft [15 x 15 m]). This is mainly due to the underlying features of HEC-RAS 2D, which consider:

- Volume rating curves of each cell based on detailed LiDAR data within footprint of each cell
- Profile along all faces of the cell based on detailed LiDAR data

Each cell in the model is assigned a Manning's roughness coefficient. Roughness coefficients are assigned using land cover categories. Several sources report Manning's roughness coefficients for flow over various surfaces. The following three sources are referenced here:

- The U.S. Department of Agriculture Natural Resources Conservation Service's Technical Release 55 (TR-55) (USDA-SCS, 1986)
- HEC-RAS Reference Manual (USACE, 2020)
- Open-Channel Hydraulics (Chow, 1988)

The references listed above provide a range of acceptable values for Manning's roughness coefficients for overland floodplain flow. Because flow is generally expected to be directed away from the NI/CI area, a higher Manning's roughness coefficient generally results in higher water levels. This conclusion is based on Manning's equation, which indicates that an increased roughness (“n”) will result in slowed velocities and an increased hydraulic radius (“R”), leading to deeper flow. Roughness associated with each land use type is presented in Figure 2.4.3-35.

$$V = \frac{1.49}{n}\ R^{2/3}\, S^{1/2}$$

(Equation -2.4-1)

where:

V = velocity (ft/s)

n = the Manning's roughness coefficient (dimensionless)

S = the topographic slope in the direction of flow (ft/ft)

R = the hydraulic radius (ft)

R = A/P (flow area [ft^2]/wetted perimeter [ft])

The main model control parameters are provided in Figure 2.4.3-40. The computation interval is used in the unsteady flow calculations. This is one of the most important parameters entered into the model. Choosing this value should be done with care and consideration as to how it will affect the simulation. The computation interval should be based on several factors. First, the interval should be small enough to accurately describe the rise and fall of the hydrographs being routed. A general rule of thumb is to use a computation interval that is equal to or less than the time of rise of the hydrograph divided by 20 (USACE, 2020). A second way of computing the appropriate time step is by applying a numerical accuracy criterion called the Courant condition. The Courant condition criteria looks at cross section spacing and flood wave velocity. The basic premise is that the computational interval should be equal to or less than the time it takes water to travel from one cross section to the next. A detailed description of the Courant condition can be found in the HEC-RAS2D user manual (HEC-RAS2D, 2020). Use of a time step based on the Courant condition will give the best numerical solution, but it may cause the model to take a lot longer to run. In this analysis, sensitivity analysis showed that a 20-s computation interval results in a stable model and captures the hydraulics of the PMP hyetograph.

The following main hydraulic structures were identified along West Coloma Creek during site survey and considered in the model as presented in Figure 2.4.3-30 and Figure 2.4.3-37:

- Whitley Road: three reinforced concrete pipe culverts, each 4 ft (1.2 m) in diameter
- Highway 35: Five reinforced concrete box culverts, each 5 ft by 5 ft (1.5 m by 1.5 m)
- Jesse Rigby Road: Four reinforced concrete box culverts, each 8 ft by 8 ft (2.4 m by 2.4 m)
- Farmland FM 2235: A road crossing exists but is considered as blocked. This is conservative as the road crossing is downstream of the site.

Due to the intensity of PMP event and small size of West Coloma Creek compared with the flat floodplain, sensitivity analysis showed that hydraulic structures do not impact the results when 2D modeling is being used. Brake lines were added to the top of roads and basin embankments to capture their topography in the model. Basins 31 and 5 (see Figure 2.4.3-39) are considered to be at their normal operating level during PMP and they are not overtopped.

Water surface elevations, depths, and velocities around the LMGS site were investigated and velocities near edges of the site were compared to permissible velocities published in a comprehensive literature review performed by the U.S. Army Engineer Research and Development Center as part of the Ecosystem Management and Restoration Research Program (USACE, 2001).

Figure 2.4.3-41 and Figure 2.4.3-42 present the maximum water depth and water surface elevation for the entire 2D model domain and for the area focused on the site, respectively. Model results show that the maximum water elevation around the site is approximately 31 ft (9.4 m) NAVD 88. This elevation is on the north-western and eastern boundaries of the site. Figure 2.4.3-43 presents the water surface elevation time series for a location at the northern edge of the LMGS site. This figure shows that the water surface elevation is less than 31 ft (9.4 m). Minor inundation shown on Figure 2.4.3-42 at the edges of the site is due to direct rainfall on the site itself that will be drained with proper site grading.

Figure 2.4.3-44 and Figure 2.4.3-45 present the maximum velocity for the entire 2D model domain and for the area focused on the site, respectively. Maximum velocity occurs on the north-eastern corner of the site with an approximate velocity of 2 fps (0.6 m/s). According to Table 2 of USACE, 2001, the critical velocity for a 6-in. (15-cm) gravel is 4 fps (1.2 m/s) and for 2-in. (5-cm) gravel is 3 fps (0.9 m/s). Therefore, it is necessary to protect the berm around the site with boulders of medium to large size with proper geotextile and grading underneath to protect against scouring. Outside the footprint of the site velocity is around 2 fps (0.6 m/s) and because the area is covered with vegetation, erosion is not expected as the permissible velocity for short native and bunch grass is 3 fps (0.9 m/s) per Table 2 of USACE, 2001.V

2.4.3.2.6 Coincident Wind Activity

Wind-driven waves were calculated using the “Wind Speed Adjustment and Wave Growth” module of the Automated Coastal Engineering System (ACES) in the Coastal Engineering Design & Analysis System (CEDAS) Version 4.03 (CEDAS-ACES).

Inputs to the ACES program included wind fetch option shallow restricted, elevation of observed wind, observed wind speed, duration of observed wind, duration of final wind, latitude of observation, restricted fetch geometry, and average fetch depth. ACES allows for the input of duration for both observed and final wind speed because it can convert between wind speed durations internally. For this calculation, the ACES inputs for both were the same because the wind speed durations were converted previously (prior to input).

The shallow restricted option was chosen because the waves are not expected to propagate under a deep-water condition for significant duration, and the fetch is not unlimited for wave formation; selecting the shallow restricted option therefore yields more accurate results. Outputs from the ACES program were wave height (H_{mo}) and wave period (T_p). The fetch geometry and average depth along the fetch were determined using the outputs of HEC-RAS 2D as presented in Figure 2.4.3-46. Average water depth along the fetches was calculated by obtaining the raster water depth information along the fetch line and then averaging these values. Various fetches with separation angles of 25 degrees were evaluated around the site. Fetch 12 is selected as the controlling fetch as it has the second longest fetch length and water depth. Fetch 11 is approximately 140 ft (42.7 m) longer than Fetch 12, but its water depth is about 0.8 ft (0.2 m) lower than Fetch 12.

Wind approaching along Fetch F12 (see Figure 2.4.3-46) is modeled in ACES to determine the controlling (maximum) significant wave height. The equations and calculation flow logic for the ACES formulations are included in the reference manual (USACE, 1992).

In accordance with Equation 2-2 of EM 1110-2-1614, (USACE, 1995), reproduced below, and ANSI/ANS-2.8-1992 (ANSI/ANS, 1992), the design wave height, also referred to herein as $H_{1\%}$ (the average wave height of the highest 1 percent of waves), used for wave runup and the calculation of wave effects was calculated using the following approximation.

$$H_{1\%} \approx 1.67\, H_s$$

(Equation 2.4-2)

Where:

H_s = significant wave height at embankment toe

Note that the significant wave height represents the average wave height of the highest 1/3 of the waves.

Wind information was obtained from ANSI/ANS-2.8-1992, Figure 1 (ANSI/ANS, 1992), as an annual extreme-mile wind speed of 50 mph (80 kph) at the LMGS site. This value represents a 2-yr. mean recurrence interval at 30 ft (9 m) above ground. The annual extreme-mile wind speed was converted to a 1-hr duration wind speed using the Coastal Engineering Manual (CEM), Figures II-2-1 and II-2-2 (USACE, 1995). The wind speeds were then converted to 10-, 15-, and 20-minute wind speed durations (as appropriate for wave generation for the applied fetches) using CEM Figure II-2-1.

Wind setup was calculated using U.S. Bureau of Reclamation ACER Technical Memorandum No. 2 (USBR, 1981), Equation 4, reproduced below:

$$S = {U^2\, F}/{1400\, D}$$

(Equation 2.4-3)

Where:

S = wind setup in ft

U = design wind velocity in mph

F = wind fetch in mi

D = average water depth in ft

The calculated value was added to the PMF water surface elevation (WSEL) for wave runup calculation. Results of the calculation of wind-driven waves, wind setup, and conversion to $H_{1\%}$

are listed in Table 2.4.3-23. The wave height was calculated with an $H_{1\%}$ of 1.19~~6.31~~ ft (~~1.92~~0.36 m).

West Coloma Creek is not a gaged stream and there is no USGS gage along the creek. The creek is dry most of the time and no constant flow runs through the Creek. It has been manually developed to drain excess water out of the croplands to Matagorda Bay. Therefore, no base-flow is considered in this analysis.

2.4.3.2.7 PMF Water Level over West Coloma Creek

Model results show that the maximum water depth around the site is approximately 31 ft (9.4 m) NAVD 88. The vertical extent of the runup on the embankment was calculated to be 33.56 ft (10.23 m) NAVD 88. This indicates that the total water level will conservatively be 33.56 ft (10.23 m) NAVD 88, which is 2 ft (0.6 m) above finish grade elevation of 31.5 ft (9.6 m).

2.4.3.3 Additional PMF Analysis on Guadalupe and San Antonio Watersheds

Two independent analyses were performed. The first analysis is the PMF over Guadalupe River Basin combined with 500-year flow from San Antonio River Basin. The second analysis is the PMF over San Antonio River Basin combined with 500-year flow from Guadalupe River Basin.

The USACE provided HEC-HMS and HEC-RAS models for the Guadalupe River Basin. A comprehensive assessment verified, validated, and updated these models to serve as the base framework. The final calibrated HEC-HMS and HEC-RAS models were utilized for this study. The extent of the models is from the most upstream point of the basin in Kerr County, Texas, at approximate river mile 230 to Mission Lake in Calhaun County, Texas, at approximate river mile 0 for the Guadalupe River Basin shown in Figure 2.4.3-47.

An independent HEC-HMS model is developed for San Antonio River Basin. The model extends from the most upstream point in Baxter County, Texas, at approximate river mile 2342 to the Guadalupe River in Calhaun County, Texas, at approximate river mile 7 of the Guadalupe River. The extent of the modeled reach is shown in Figure 2.4.3-48. A full calibration of the San Antonio HEC-HMS model was performed. San Antonio River Basin HEC-HMS model generated PMF flow that was used as lateral flow on the same HEC-RAS model developed for the Guadalupe River Basin.

National Oceanic and Atmospheric Administration (NOAA/NWS) HMR 51 (NWS, 1978) and HMR 52 (NWS, 1992) are used to determine the probable maximum precipitation (PMP) over both Guadalupe and San Antonio Basins. Basin-wide average rainfall values derived from the HMRs were compared with those developed in a Texas Commission on Environmental Quality (TCEQ) study in 2016 (TCEQ, 2016). The HMR basin average values were higher than those from TCEQ. Therefore, it is expected that the PMF values derived from HMR PMPs are bounding. The PMP values from HMR 51 and 52 were used to perform the PMF analysis described in this subsection.

Co-incident wind-wave activity was analyzed considering the 2-yr. mean recurrence interval annual extreme-mile wind speed of 50 mph (80 kph) obtained from ANSI/ANS 2.8-1992 (ANSI/ANS, 1992). Maximum wave runup is estimated to be 10.83 ft (3.30 m) for Guadalupe River PMF and 9.52 ft (2.9 m) for San Antonio River PMF.

Maximum wind setup is estimated to be 0.79 ft (0.24 m) for Guadalupe River PMF and 0.68 ft (0.21 m) for San Antonio River PMF. Sea level rise at the end of the project life is estimated to be 1.18 ft (0.35 m).

The maximum PMF still water level of the Guadalupe River at the LMGS site, before wind-wave induced setup and run-up, is predicted to be at elevation 28.0 ft (8.53 m) NAVD 88. Adding the combined maximum wind setup, wave run-up predictions and sea level rise, the maximum Gudalupe River PMF flooding water level at the LMGS site is postulated to be at elevation 40.80 ft (12.44 m) NAVD 88.

The maximum PMF still water level of the Guadalupe River with PMP over San Antonio River Basin at the LMGS site, before wind-wave induced setup and run-up, is predicted to be at elevation 26.0 ft (7.92 m) NAVD 88. Adding the combined maximum wind setup, wave run-up predictions and sea level rise, the maximum San Antonio River PMF flooding water level at the LMGS site is postulated to be at elevation 37.38 ft (11.39 m) NAVD 88.

A barge canal is located between the Guadalupe River and LMGS site with embankment elevation at 41.72 ft. (12.71 m) NAVD88. Therefore, this embankment is not inundated with a margin of 11 in. (28 cm).

This is lower than the 41.72 ft (12.7 m) NAVD 88 by 0.92 ft (0.28 m). Therefore, the left bank of the river will not be overtopped by wind-wave runup effect. The site will not be flooded during Guadalupe River PMF event.

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Table 2.4.3-1
All-Season PMP Precipitation Depths in Inches for the Guadalupe River Basin

Area (mi^2)	PMP (in.)				
	6-hr.	12-hr.	24-hr.	48-hr.	72-hr.
10	32.0	38.7	47.1	51.8	55.7
200	24.6	31.2	39.5	44.3	48.8
1000	18.2	24.9	33.2	37.7	41.3
5000	10.1	15.0	21.9	26.6	30.7
10,000	7.6	11.8	17.6	22.5	26.5
20,000	5.6	9.2	13.6	18.0	22.0

Table 2.4.3-2
PMF Basin Runoff Model Parameters

Model Sub-Basin[a]	Drainage Area (mi^2)	Initial Loss (in.)	Constant Loss Rate (in/hr)	Synder's Lag[b] (hr.)	Snyder's C_p[b]	Base Flow		
						STRTQ[b] (cfs/mi^2)	QRCSN[b]	Recession Coefficient[b]
1	179.14	1.0	0.15	(c)	(c)	(d)	(d)	(d)
2	102.00	1.0	0.15	(c)	(c)	(d)	(d)	(d)
3	31.69	1.0	0.15	(c)	(c)	(d)	(d)	(d)
4	131.82	1.0	0.15	(c)	(c)	(d)	(d)	(d)
5	382.53	1.0	0.15	(c)	(c)	(d)	(d)	(d)
6	473.43	1.0	0.15	(c)	(c)	(d)	(d)	(d)
8	64.07	1.0	0.15	(c)	(c)	(d)	(d)	(d)
9	32.04	1.0	0.15	(c)	(c)	(d)	(d)	(d)
10	20.14	0.0	0.0	(c)	(c)	(d)	(d)	(d)
11	86	1.0	0.1	2.5	0.6	2	0.05	0.887
12	130	0.5	0.1	4.0	0.625	0.4	0.08	0.887
13	456	0.5	0.1	8.0	0.55	0.3	0.05	0.887
14	355	0.5	0.05	2.0	0.625	6	0.1	0.92
15A	57	1.0	0.05	2.0	0.6	0.3	0.1	0.887
15B	24	1.0	0.05	2.0	0.625	0.3	0.1	0.887
16	48.5	1.0	0.05	2.5	0.6	0.3	0.05	0.788
17	46.5	1.0	0.05	2.5	0.6	0.3	0.05	0.788
18	82	1.0	0.05	3.5	0.6	0.3	0.05	0.788
19	143	1.0	0.05	3.5	0.6	0.3	0.05	0.788
20	82	1.0	0.05	5.5	0.6	0.3	0.05	0.788
21	23	1.0	0.05	2.3	0.6	0.3	0.1	0.887
22A	112	0.5	0.05	4.2	0.6	0.1	0.24	0.788
22B	277	0.5	0.05	5.0	0.6	0.1	0.1	0.788
23	108	0.5	0.05	5.0	0.6	0.1	0.05	0.887
24	69	0.5	0.05	3.5	0.6	0.3	0.05	0.887
25	483	0.5	0.05	22	0.55	0.1	0.6	0.887
26	209	0.5	0.05	12.5	0.55	0.3	0.05	0.887
27A	390	1.0	0.05	37.0	0.75	0.1	0.05	0.75
27B	159	1.0	0.05	37.0	0.75	0.1	0.05	0.75
27C	162	1.0	0.05	34.0	0.75	0.3	0.05	0.75
28A	132.5	0.5	0.05	12.5	0.55	0.1	0.05	0.887
28	132	0.5	0.05	12.5	0.55	0.1	0.05	0.887
29	357	1.0	0.05	4.2	0.52	0.1	0.1	0.75
30	133	1.0	0.05	5.0	0.58	0.1	0.01	0.75
31	148	1.0	0.05	5.0	0.55	0.1	0.01	0.75

a) Subbasin 7 is intentionally omitted.
b) No base flow is assumed by USACE (USACE, 2005a).
c) Use unit hydrographs (Table 2.4.3-13).
d) Definitions are given in HEC-HMS Users Manual (USACE, 2006a).

Table 2.4.3-3
Largest Five Recorded Peak Discharges for USGS Gage No. 08167000, Guadalupe River at Comfort, Texas

Water Year	Date	Peak Discharge (cfs)
1978	August 2, 1978	240,000
1900	July 16, 1900	182,000[a]
1935	June 14, 1935	148,000
1987	July 17, 1987	130,000[b]
2002	July 4, 2002	128,000[b]

a) Discharge is a Historic Peak.
b) Discharge affected to unknown degree by Regulation or Diversion.
Source: (USGS, 2023a), the drainage area is 839 mi^2.

Table 2.4.3-4
Largest Five Recorded Peak Discharges for USGS Gage No. 08167500, Guadalupe River near Spring Branch, Texas

Water Year	Date	Peak Discharge (cfs)
1978	August 3, 1978	160,000
1932	July 3, 1932	121,000
1997	June 2, 1997	116,000
1935	June 15, 1935	114,000
2002	July 5, 2002	94,400

Source: (USGS, 2023b) and the drainage area is 1315 mi^2.

Table 2.4.3-5
Largest Five Recorded Peak Discharges for USGS Gage No. 08176500, Guadalupe River at Victoria, Texas

Water Year	Date	Peak Discharge (cfs)
1999	October 20, 1998	466,000[(a)]
1936	July 3, 1935	179,000[(a)]
1981	September 2, 1981	105,000[(a)]
2005	November 26, 2004	102,000[(a)]
2017	August 20, 2017	86,500[(a)(b)]

a) Discharge affected by Regulation or Diversion.
b) Discharge due to snowmelt, hurricane, ice-jam or debris dam breakup.
Source: (USGS, 2023c); the drainage area is 5198 mi^2.

Table 2.4.3-6
Subbasin Drainage Areas and NWS Rainfall Used in Basin Runoff Model Calibration

Model Subbasin	Drainage Area (mi^2)	NWS Subbasin Rainfall File Name[(a)]
11	86	NBRT2
12	130	GBCT2
13	456	Weighted Average (SEGT2, GNLT2U, & GNLT2M)[(b)]
14	355	Weighted Average (WMBTU & WMBT)[(b)]
15A	57	KYET2
15B	24	LLGT2U
16	48.5	LLGT2U
17	46.5	LLGT2U
18	82	LLGT2U
19	143	LLG2T
20	82	LLG2T
21	23	GNLT2
22A	112	LULT2U
22B	277	LULT2
23	108	GNLT2
24	69	CUET2U
25	483	DLWT2
26	209	CUET2U
27A	390	Weighted Average (WHOT & WHOT2U)[(b)]
27B	159	WHOT2M
27C	162	CUET
28A	132.5	VICTU
28	132	VICT2
29	357	SCDT2
30	133	CKDT2
31	148	DUPT2

a) NWS subbasin names as defined in Figure 2.4.3-4.
b) Weighted average is based on drainage areas.

Table 2.4.3-7
Subbasin Runoff Parameters 1998 Calibration Results

Model Sub-Basin	Initial Loss (in.)	Constant Loss Rate (in/hr)	Synder's Lag[a] (hr.)	Synder's C_p[a]	Base Flow		
					STRTQ[a] (cfs/mi^2)	QRCSN[a]	Recession Coefficient[a]
11	1.5	0.15	3.0	0.6	0.3	0.02	0.887
12	0.5	0.15	5.0	0.625	0.4	0.08	0.887
13	0.5	0.10	10	0.5469	0.3	0.05	0.887
14	1.8	0.45	2.7	0.6	0.05	0.04	0.887
15A	1.0	0.08	2.0	0.6	0.05	0.01	0.887
15B	1.0	0.08	2.0	0.6	0.05	0.1	0.887
16	1.9	0.10	3.0	0.6	0.05	0.01	0.750
17	1.9	0.10	3.0	0.6	0.05	0.01	0.750
18	2.5	0.10	4.0	0.6	0.05	0.01	0.750
19	2.0	0.10	4.0	0.6	0.05	0.01	0.700
20	2.0	0.10	6.0	0.6	0.05	0.01	0.750
21	2.0	0.10	2.5	0.5469	0.05	0.1	0.887
22A	1.65	0.05	5.0	0.6	0.01	0.04	0.788
22B	1.5	0.05	6.0	0.6	0.01	0.04	0.788
23	0.8	0.10	6.0	0.6	0.1	0.05	0.887
24	0.5	0.08	4.0	0.6	0.3	0.05	0.887
25	0.5	0.11	25	0.5	0.1	0.6	0.1
26	0.5	0.05	15	0.5	0.3	0.05	0.887
27A	1.5	0.10	44.0	0.75	0.1	0.001	0.55
27B	1.5	0.10	43.0	0.75	0.1	0.001	0.55
27C	1.5	0.10	40.0	0.75	0.1	0.001	0.55
28A	0.5	0.10	15.0	0.5469	0.1	0.05	0.887
28	0.5	0.10	15.0	0.5469	0.1	0.05	0.887
29	2.5	0.12	14.0	0.55	0.1	0.1	0.75
30	2.0	0.12	6.0	0.55	0.1	0.01	0.75
31	2.0	0.12	6.0	0.55	0.1	0.01	0.75

a) Definitions are given in HEC-HMS Users Manual (USACE, 2006a).

Table 2.4.3-8
Channel Elements — Muskingum K and X 1998 Calibration Results

Model Channel Elements	Channel Reach Location[a]	Muskingum Channel Routing		
		K (hr.)	X	Number of Sub-Reaches
SMCNRB	thro' Sub 11	3	0.2	3
NBRSMR	thro' Sub 13	30	0.35	20
WMBKYE	thro' Sub 15A	-	-	-
KYESMR	thro' Sub 15B	-	-	-
SCSBLC	[b]	-	-	-
BLCYRK	thro' Sub 18	6	0.3	5
YRKLLG	thro' Sub 20	3.3	0.2	2
LLGPLM	thro' Sub 21	4	0.1	2
LCPSM	thro' Sub 22B	25	0.3	10
PLMGR	thro' Sub 23	10	0.3	5
SMRGNL	[c]	2	0.3	1
GNLPCH	thro' Sub 24	10	0.3	6
PCHSAN	thro' Sub 26	22	0.3	11
WHOGR	thro' Sub 27C	20	0.4	7
SANCUE	[d]	2	0.2	1
UPPERVIC	thro' Sub 28A	4	0.4	9
CUEVIC	thro' Sub 28	4	0.4	9
Vic-Coleto	[e]	3	0.3	3
Coleto-1	thro' Sub 30	7	0.25	3
Coleto-2	thro' Sub 31	7	0.25	3
Coleto-Vic	[f]	1	0.2	1

a) Refer to Figure 2.4.3-3.
b) On San Marcos River below SCS #3 and 5 reservoirs to its confluence with Blanco River.
c) On Guadalupe River below its confluence with San Marcos River to USGS Gage at Gonzales, Texas.
d) On Guadalupe River below its confluence with Sandies Creek to USGS Gage at Cuero, Texas.
e) On Guadalupe River below USGS Gage at Victoria, Texas.
f) On Coleto Creek below Sub 31 to its confluence with Guadalupe River.

Table 2.4.3-9
Subbasin Parameters 2004 Calibration Results

Model Sub-Basin	Initial Loss (in.)	Constant Loss Rate (in/hr)	Synder's Lag[(a)] (hr.)	Synder's C_p[(a)]	Base Flow STRTQ[(a)] (cfs/mi^2)	Base Flow QRCSN[(a)]	Base Flow Recession Coefficient[(a)]
11	1.3	0.11	4.0	0.6	2	0.05	0.887
12	0.5	0.15	5.0	0.625	0.4	0.08	0.887
13	0.5	0.10	13.0	0.5469	0.3	0.05	0.887
14	0.42	0.09	3.8	0.625	6	0.1	0.92
15A	2.0	0.20	6.0	0.6	0.3	0.1	0.887
15B	1.0	0.08	3.0	0.625	0.3	0.1	0.887
16	1.8	0.05	4.0	0.6	0.3	0.05	0.788
17	1.8	0.05	4.0	0.6	0.3	0.05	0.788
18	2.5	0.05	5.0	0.6	0.3	0.05	0.788
19	2.0	0.05	5.0	0.6	0.3	0.05	0.788
20	2.0	0.05	8.0	0.6	0.3	0.05	0.788
21	1.8	0.08	3.0	0.6	0.3	0.05	0.887
22A	0.5	0.08	5.8	0.35	0.1	0.24	0.600
22B	0.5	0.15	6.0	0.6	0.1	0.1	0.788
23	0.8	0.10	6.0	0.5469	0.1	0.05	0.887
24	0.5	0.08	6.0	0.5469	0.3	0.05	0.887
25	1.5	0.15	20	0.55	0.1	0.6	0.887
26	0.5	0.05	8.0	0.5469	0.3	0.05	0.887
27A	1.55	0.12	43.0	0.6	0.1	0.05	0.75
27B	1.55	0.12	35.0	0.6	0.1	0.05	0.75
27C	1.55	0.12	33.0	0.6	0.3	0.05	0.75
28A	2.5	0.15	8.0	0.5469	0.1	0.05	0.887
28	2.5	0.15	8.0	0.5469	0.1	0.05	0.887
29	1.8	0.09	5.0	0.52	0.1	0.07	0.75
30	1.5	0.10	6.0	0.58	0.1	0.01	0.75
31	1.5	0.10-	6.0	0.55	0.1	0.01	0.75

a) Definitions are given in HEC-HMS Users Manual (USACE, 2006a).

Table 2.4.3-10
Channel Elements — Muskingum K and X Values 2004 Calibration Results

Model Channel Elements	Channel Reach Location[a]	Muskingum Channel Routing		
		K (hr.)	X	Number of Sub-reaches
SMCNRB	thro' Sub 11	4	0.25	3
NBRSMR	thro' Sub 13	39	0.2	20
WMBKYE	thro' Sub 15A	-	-	-
KYESMR	thro' Sub 15B	-	-	-
SCSBLC	(b)	-	-	-
BLCYRK	thro' Sub 18	8	0.15	5
YRKLLG	thro' Sub 20	4	0.25	2
LLGPLM	thro' Sub 21	4	0.1	2
LCPSM	thro' Sub 22B	20	0.2	10
PLMGR	thro' Sub 23	15	0.2	5
SMRGNL	(c)	2	0.2	1
GNLPCH	thro' Sub 24	12	0.2	6
PCHSAN	thro' Sub 26	22	0.2	11
WHOGR	thro' Sub 27C	18	0.2	7
SANCUE	(d)	2	0.15	1
UPPERVIC	thro' Sub 28A	18	0.2	9
CUEVIC	thro' Sub 28	18	0.2	9
Vic-Coleto	(e)	2	0.2	3
Coleto-1	thro' Sub 30	3	0.25	3
Coleto-2	thro' Sub 31	3	0.25	3
Coleto-Vic	(f)	1	0.2	1

a) Refer to Figure 2.4.3-3.
b) On San Marcos River below SCS #3 and 5 reservoirs to its confluence with Blanco River.
c) On Guadalupe River below its confluence with San Marcos River to USGS Gage at Gonzales, Texas.
d) On Guadalupe River below its confluence with Sandies Creek to USGS Gage at Cuero, Texas.
e) On Guadalupe River below USGS Gage at Victoria, Texas.
f) On Coleto Creek below Sub 31 to its confluence with Guadalupe River.

Table 2.4.3-11
PMF Basin Runoff Model Muskingum Channel Routing Coefficients K and X

Model Channel Elements[(a)]	Muskingum Channel Routing		
	K (hr.)	X	Number of Sub-reaches
NFGage	(b)	(b)	(b)
HNTJNC	(b)	(b)	(b)
GRAJNC	(b)	(b)	(b)
JNCCOM	(b)	(b)	(b)
COMFORT	(b)	(b)	(b)
COMFCAN	(b)	(b)	(b)
SMCNRB	3	0.2	3
NBRSMR	30	0.35	20
WMBKYE	-	-	-
KYESMR	-	-	-
SCSBLC	-	-	-
BLCYRK	6	0.3	5
YRKLLG	3.3	0.2	2
LLGPLM	4	0.1	2
LCPSM	25	0.3	10
PLMGR	10	0.3	5
SMRGNL	2	0.3	1
GNLPCH	10	0.3	6
PCHSAN	22	0.3	11
WHOGR	20	0.4	7
SANCUE	2	0.2	1
UPPERVIC	4	0.4	9
CUEVIC	4	0.4	9
Vic-Coleto	3	0.3	3
Coleto-1	3	0.2	3
Coleto-2	3	0.2	3
Coleto-Vic	1	0.2	1

a) Refer to Tables 2.4.3-8, 2.4.3-10, and 2.4.3-14 for locations of the channel reaches.
b) Refer to Table 2.4.3-14.

Table 2.4.3-12
Canyon Dam Watershed Subbasins

Subbasin	Descriptions	Drainage Area (mi^2)
1	North Fork of Guadalupe River	179.14
2	South Fork of Guadalupe River	102.00
3	Area between North & South Forks of Guadalupe and mouth of Johnson Creek	31.69
4	Johnson Creek	131.82
5	Area between Johnson/Guadalupe confluence to Comfort	382.53
6	Area between Comfort and Head of Canyon Lake	473.43
8	Area adjacent to north side of Canyon Lake	64.07
9	Area adjacent to south side of Canyon Lake	32.04
10	Canyon Lake Surface	20.14
	Total	1417.85

Source: (USACE, 2005a)
Note: Subbasin 7 is intentionally omitted.

Table 2.4.3-13
Canyon Dam Watershed 1-Hour Unit Hydrographs for Subbasins
(Sheet 1 of 2)

Time (hr.)	Subbasin Hydrographs (cfs)								
	1	2	3	4	5	6	8	9	10
0	45	235	357	300	5	2	19,049	12,120	12,995
1	270	1215	11,644	1560	2275	47	16463	7347	12,995
2	1530.0	2979	4522	3840	13,650	70	4561	1076	12,995
3	18,000	20,482	2321	26,400	60,900	512	1012	131	12,995
4	27,000	11,917	1190	15,360	40,950	1163	208	2	
5	18,000	8428	417	10,865	27,300	2558	44		
6	11,250	6311	0.0	8142	19,474	12,276	8		
7	7200	4283		5520	14,788	21,762			
8	5760	2607		3360	11,830	27,714			
9	4590	931		1200	9555	30,059			
10	3870	559		720	8008	28,272			
11	3150	470		600	6734	23,622			
12	2700	451		576	5733	17,298			
13	2250	431		552	4823	14,322			
14	1980	412		528	4095	12,276			
15	1710	392.		504	3458	10,602			
16	1440	372		480	2821	9254			
17	1260	353		456	2366	8277			
18	1080	333		432	1911	7440			
19	900	314		408	1638	6789			
20	720	294		384	1365	6185			
21	540	274		360	1138	5673			
22	360	255		336	865	5208			
23	0	235		312	637	4790			
24		216		288	455	4418			
25		196		264	273	4092			
26		176		240	182	3720			
27		157		216	91	3488			
28		137		192	73.	3302			
29		118		168	55	3023			
30		98		144	36	2790			
31		78		120	18	2604			
32		59		96	0.0	2372			

33	39	72	2186	

Table 2.4.3-13
Canyon Dam Watershed 1-Hour Unit Hydrographs for Subbasins
(Sheet 2 of 2)

Time (hr.)	Subbasin Hydrographs (cfs)								
	1	2	3	4	5	6	8	9	10
34		20		48		2000			
35				24		1814			
36						1674			
37						1535			
38						1395			
39						1256			
40						1116			
41						977			
42						884			
43						791			
44						698			
45						605			
46						512			
47						419			
48						372			
49						326			
50						279			
51						233			
52						186			
53						140			
54						93			
55						47			

Source: (USACE, 2005a)

Table 2.4.3-14
Canyon Dam Watershed Channel Elements Guadalupe River Storage-Discharge Relationships

Channel Element	Location of Channel Reach	Storage (ac-ft)	Discharge (cfs)
NF Gage	North Gauge to North and South Fork Confluence	66,112	800,000
HNTJNC	North & South Confluence to confluence with John Creek	66,112	800,000
JNCCOM	John Creek-Guadalupe confluence to Comfort	264,448	800,000
COMFORT	Comfort to a point d/s of Comfort	595,008	800,000
COMFCAN	A point d/s of Comfort to head of Canyon Lake Reservoir	330,560	800,000

Source: (USACE, 2005a)

Table 2.4.3-15
Canyon Dam Elevation-Storage-Discharge Relationship

Elevation (ft.)	Storage (ac-ft)	Discharge (cfs)
909	378,899	0.0
910	387,248	0.0
912	404,216	0.0
915	430,423	0.0
918	457,703	0.0
920	476,495	0.0
925	525719	0.0
930	578,588	0.0
935	635,221	0.0
940	695,624	0.0
943	733,602	0.0
944	746,545	2500
944.5	753,095	4750
945	759,645	7000
946	772,895	14,000
947	786,285	23,000
948	799,820	34,000
949	813,485	47,000
950	827,295	62,000
951	841,265	77,000
952	855,395	95,000
953	869,685	110,000
954	884,135	130,000
958	943,400	210,000
959	958,610	235,000
962	1,005,275	310,000
966	1,070,010	410,000
968	1,103,500	470,000
970	1,137,730	525,000
975	1,226,445	640,000

Source: (USACE, 2005a)
Note: Elevations in Table 2.4.3-15 are given in terms of National Geodetic Vertical Datum of 1929 (NGVD 29). To convert to NAVD 88, add 0.34 ft. to the values shown in the table.

Table 2.4.3-16
X and Y Coordinates of Storm Center and Optimized Orientation for PMP Estimates

Storm Center	X[a]	Y[a]	Optimized Orientation (°)
South-east corner of Subbasin 13	459.0	2599.5	155
Centroid of Subbasin 23	461.2	2607.0	155
Centroid of Subbasin 24	469.1	2598.6	155
Centroid of Subbasin 27B	433.7	2608.0	140
Lower end of Subbasin 5	373.82	2633.5	279.5[b]

a) X and Y coordinates are based on Texas State Plane system, with units in mi.
b) Not optimized; obtained from USACE (USACE, 2005a).

Table 2.4.3-17
PMF Development Starting Reservoir Water Levels

Reservoir	40% PMP Run		Full PMP Run	
	Starting Reservoir Water Level (elevation in ft.)(a)(f)	Starting Reservoir Storage (ac-ft)	Starting Reservoir Water Level (elevation in ft.)(a)(e)(f)	Starting Reservoir Storage (ac-ft)
Canyon Dam	909.0(b)	378,899(b)	947.73(b)	796,166(b)
SCS#3	611.0(c)	127(c)	613.3	190.6
SCS#5	616.2(c)	161(c)	632.8	1023
Coleto Creek Dam	98.5(d)	32,640(d)	98.9	33,863

a) Elevations in Table 2.4.3-17 are given in terms of NGVD 29. To convert to NAVD 88 for Canyon Dam, SCS #3, and SCS #5, add 0.34 ft., 0.30 ft., and 0.31 ft., respectively, to the values shown in the table.
b) (USACE, 2005a); USACE used 50% PMP as the preceding storm.
c) (USACE, 2004)
d) From HEC-HMS calibrations of the 1998 and 2004 floods.
e) These are the water levels at the respective reservoirs 3 days after the cessation of the 72-hr. 40% PMP, except as noted.
f) For Coleto Creek Dam, subtract 0.29 ft. from the values shown in the table.

Table 2.4.3-18
PMF Water Surface Profile along Guadalupe River

STA	Discharge (cfs)	Water Level (ft. NAVD 88)
7.2467	1,300,000	28.87
8.7744	1,300,000	29.97
11.1811	1,300,000	31.50
12.0915	1,300,000	32.30
12.9444	1,300,000	32.76
14.5044	1,120,000	33.01
16.0078	1,120,000	33.22
17.6557	1,120,000	33.69
18.6485	1,120,000	34.83
20.7087	1,120,000	35.92
22.0501	1,120,000	37.12
23.4397	1,120,000	39.22
25.0028	1,120,000	41.58
26.7812	1,120,000	42.95
29.5984	1,120,000	44.64
29.5984	1,120,000	44.64
30.8097	1,120,000	46.02
32.2088	1,120,000	47.73
37.1142	1,120,000	50.78
41.6305	1,100,000	54.25
46.127	1,100,000	59.42
49.2913	1,100,000	63.12
52.1817	1,100,000	66.74
54.702	1,100,000	72.70
56.1333	1,100,000	103.69
60.9682	1,100,000	104.34
63.8964	1,100,000	105.52
66.2563	1,100,000	108.32

Table 2.4.3-19
All-Season PMP Precipitation Depths in Inches for the Basin with Storm Center at the Long Mott Generating Station Site

Basin Size (mi^2)	6 hr.	12 hr.	24 hr.	48 hr.	72 hr.
10	32	38.7	47.1	51.8	55.7

Table 2.4.3-20
Depth Duration Precipitation Values for Envelope Basins in Inches with Storm Center at the Long Mott Generating Station Site

6-hr. Time Period	6 hr.	12 hr.	18 hr.	24 hr.	30 hr.	36 hr.	42 hr.	48 hr.	54 hr.	60 hr.	66 hr.	72 hr.
10-mi^2 cumulative (in.)	32	38.7	42.9	47.1	48.3	49.5	50.6	51.8	52.8	53.8	54.7	55.7
10-mi^2 increment (in.)	32	6.7	4.2	4.2	1.2	1.2	1.1	1.2	1	1	1	1
10-mi^2 increment (in/hr)	5.33	1.12	0.7	0.7	0.2	0.2	0.18	0.2	0.17	0.17	0.15	0.17

Table 2.4.3-21
Precipitation Depths in Inches for Each Realigned 6-Hour Time Period According to the Standardized Temporal Distribution from HMR 52

6-hr Increment Number	**12**	**11**	**10**	**9**	**7**	**6**	**5**	**3**	**1**	**2**	**4**	**8**
6-hr Time Period from Error! Reference source not found.	72	66	60	54	42	36	30	18	6	12	24	48
Time Period	0-6 hr.	6-12 hr.	12-18 hr.	18-24 hr.	24-30 hr.	30-36 hr.	36-42 hr.	42-48 hr.	48-54 hr.	54-60 hr.	60-66 hr.	66-72 hr.
10-mi^2 Incremental PMP (in.)	1	1	1	1	1.1	1.2	1.2	4.2	32	6.7	4.2	1.2
10-mi^2 Incremental PMP (in/hr)	0.17	0.17	0.17	0.17	0.18	0.2	0.2	0.7	5.3	1.1	0.7	0.2

Table 2.4.3-22
Curve Number Combinations

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Water	100	100	100	100
Medium Residential	57	72	81	86
Forest	30	58	71	78
Agricultural	67	77	83	87

Source: (USDA-SCS, 1986)

Table 2.4.3-23
Calculation of Wind-Driven Waves and Wind Setup Approaching the Long Mott Generating Station Site

Point of Interest	Average Depth Along Fetch (ft.)	Fetch Length (ft.)	Design Wind Speed (mph)	Wind Speed Duration (min.)	H_{mo} (ft.)	Tp (s)	Wave length (ft.)	Wind Setup (ft.)	H1% (ft.)	Wave Runup (ft.)
Embankment around the Plant	3.9	10,023	50	80	1.19	2.09	19.16	0.56	1.99	2

Figure 2.4.3-1
Guadalupe and San Antonio River Basin Stream Gages

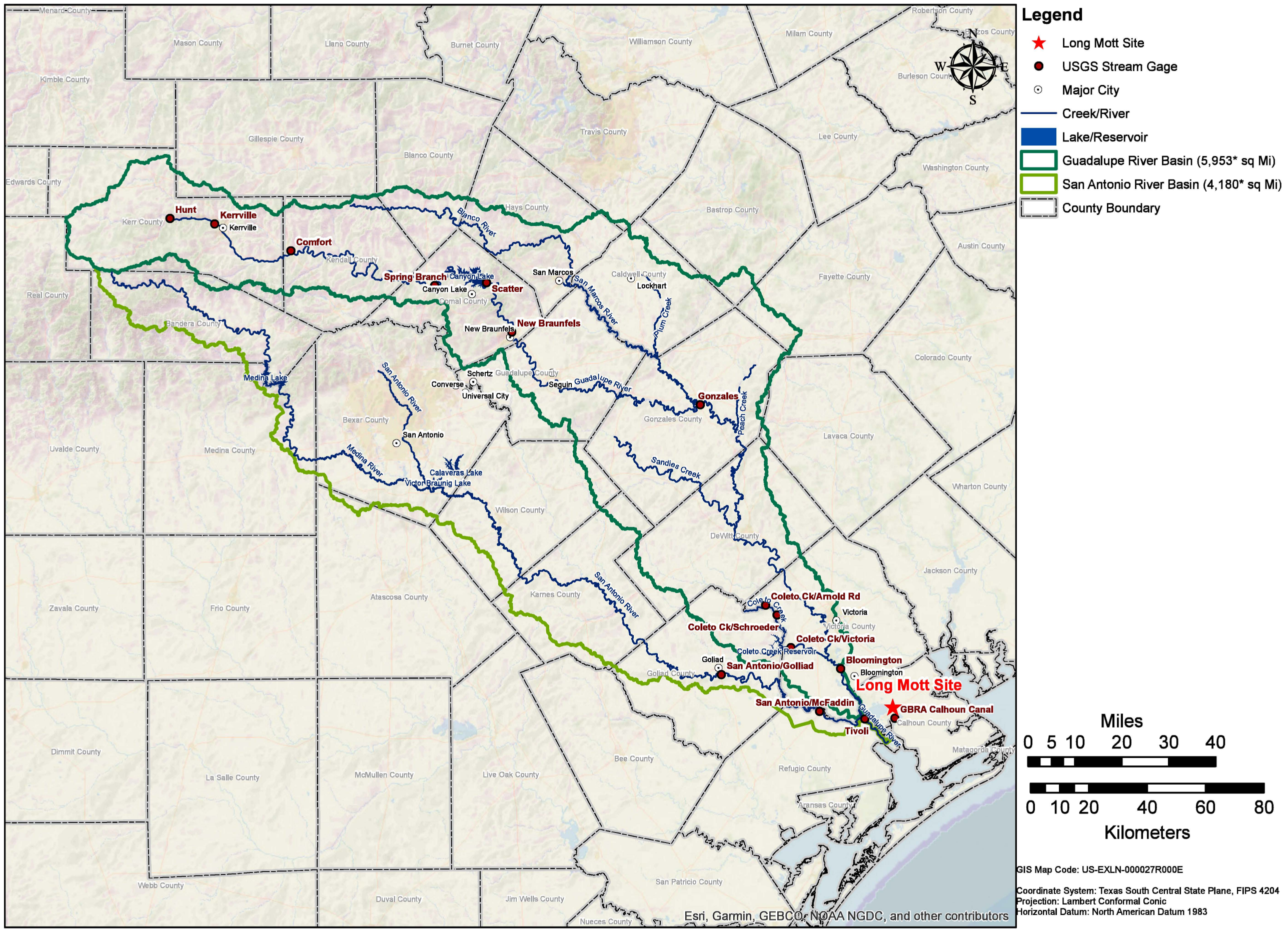


Figure 2.4.3-2
West Coloma Creek Watershed

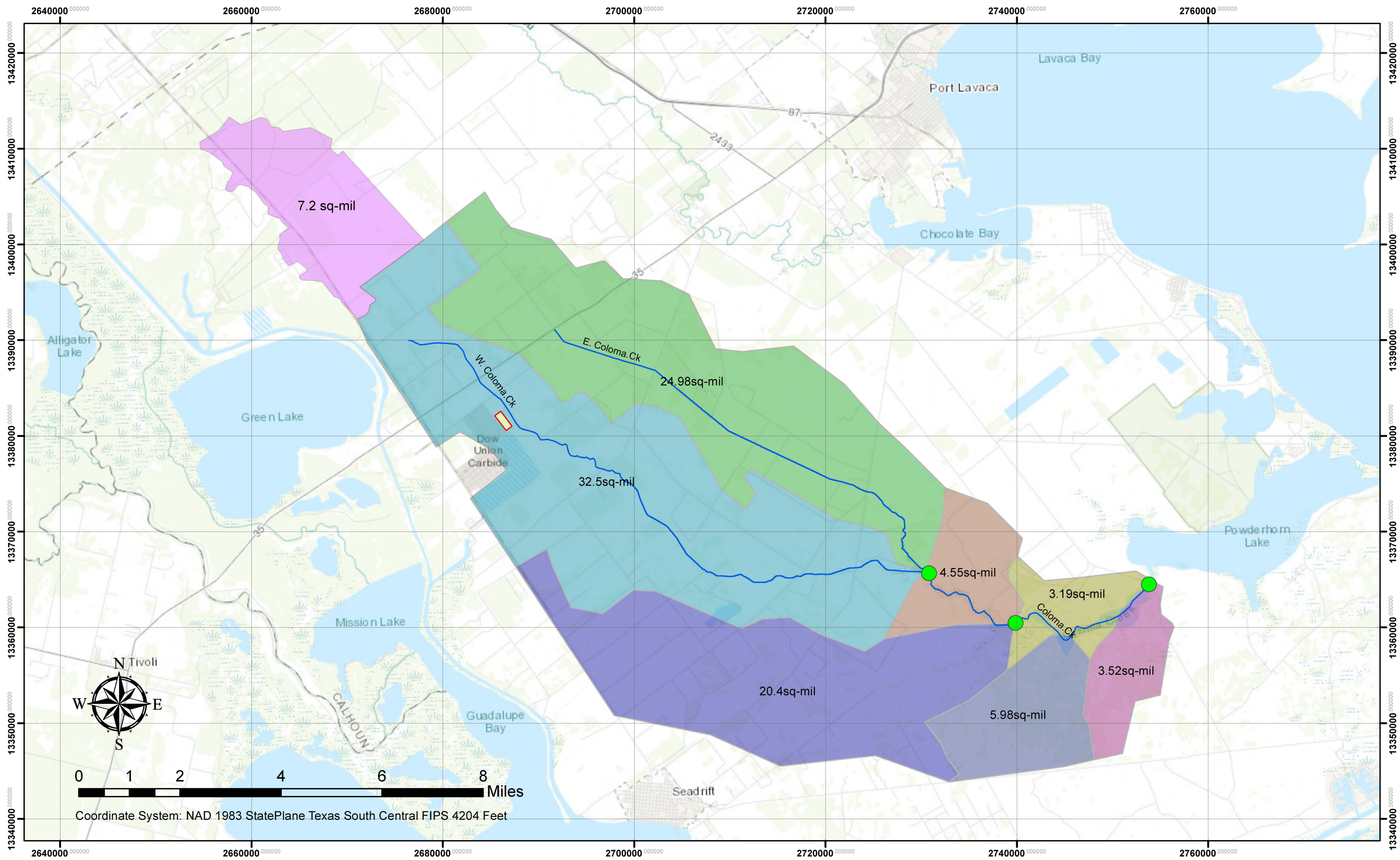


Figure 2.4.3-3
Subbasin Delineation — U.S. Army Corps of Engineers

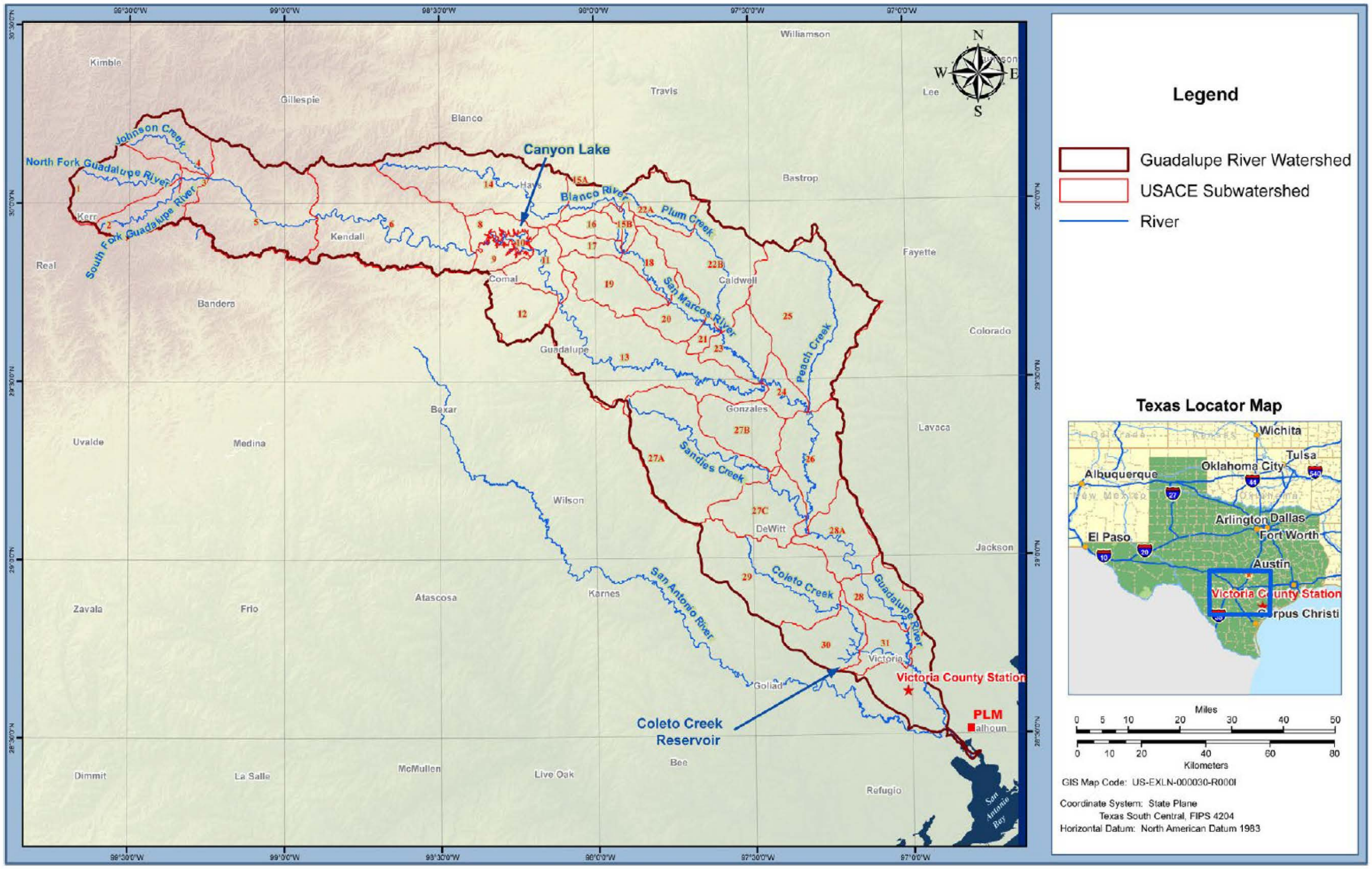


Figure 2.4.3-4
Subbasin Delineation — U.S. National Weather Service

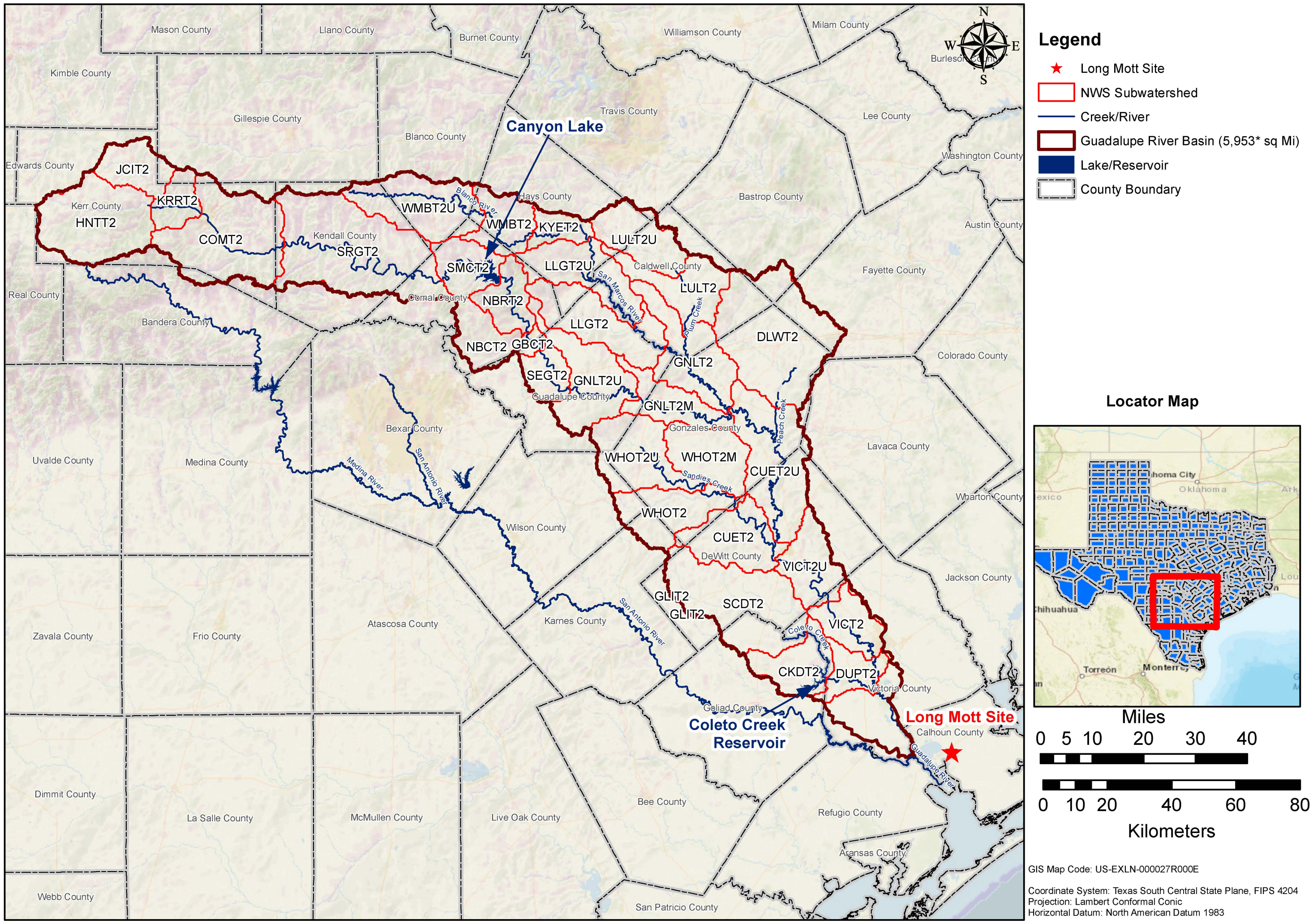


Figure 2.4.3-5
1998 Flood — Observed and Computed Hydrographs, Guadalupe River above Comal River (USGS No. 8168500)

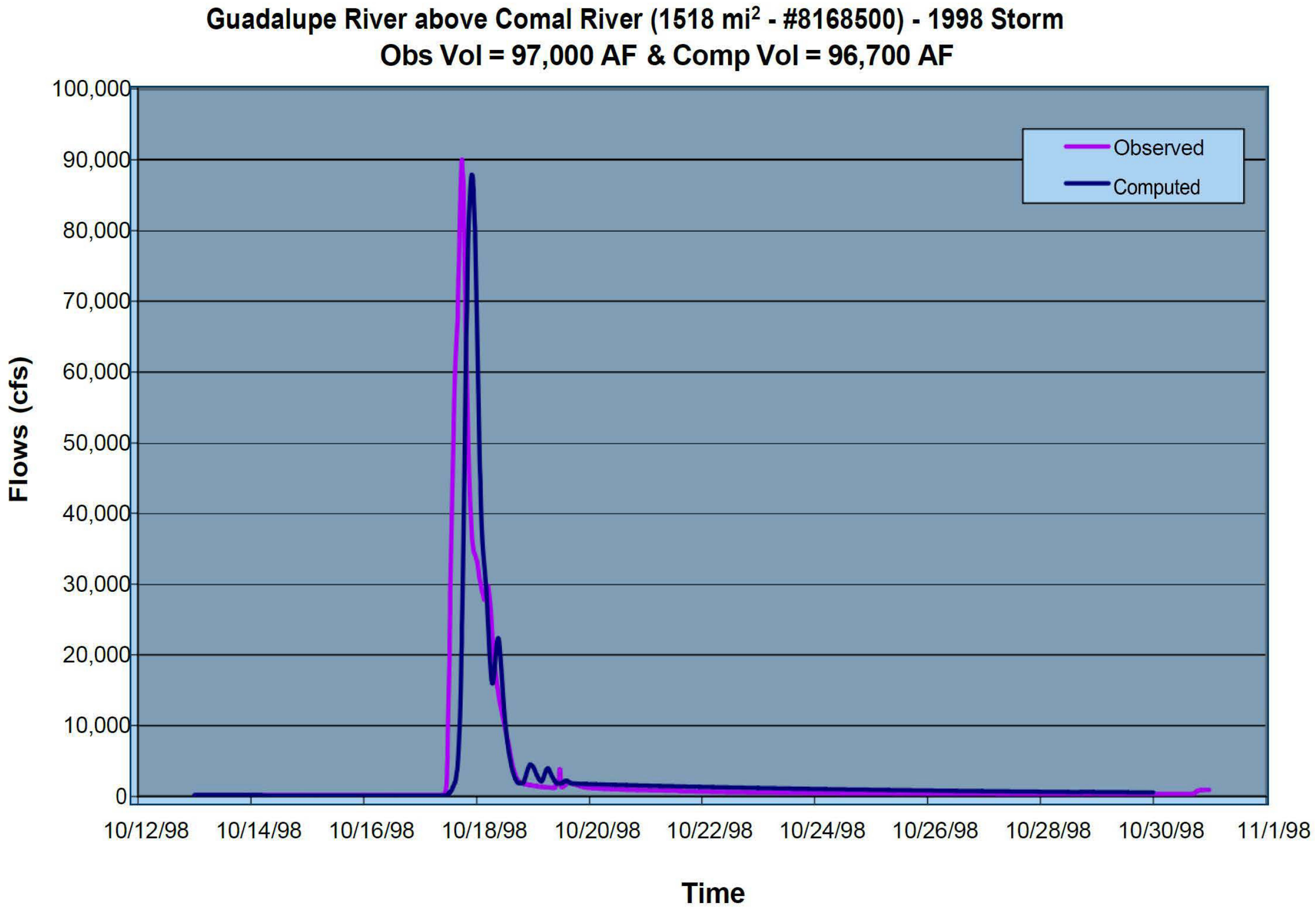


Figure 2.4.3-6
1998 Flood — Observed and Computed Hydrographs, Blanco River at Wimberley (USGS No. 8171000)

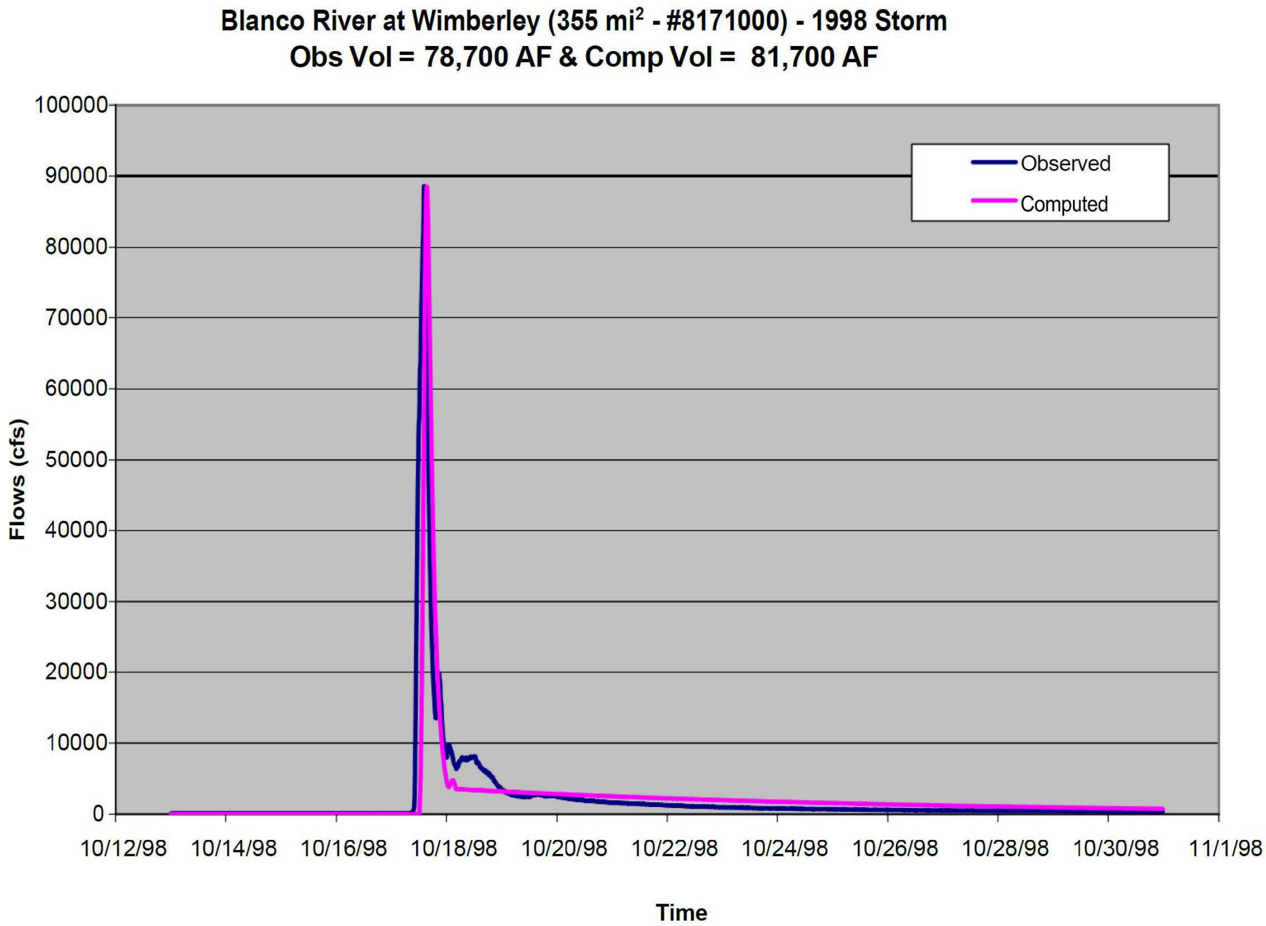


Figure 2.4.3-7
1998 Flood — Observed and Computed Hydrographs, Plum Creek at Lockhart (USGS No. 8172400)

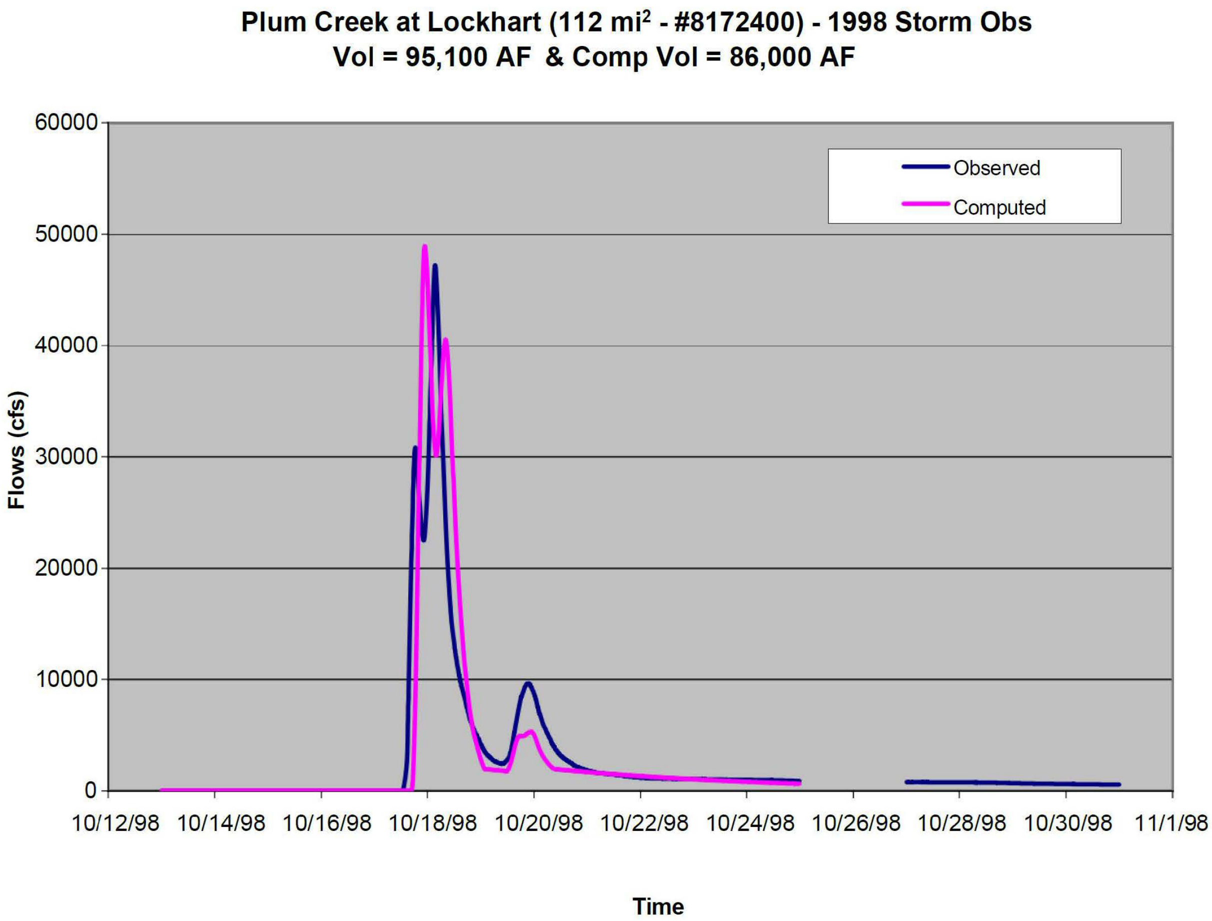


Figure 2.4.3-8
1998 Flood — Observed and Computed Hydrographs, San Marcos River at Luling (USGS No. 8172000)

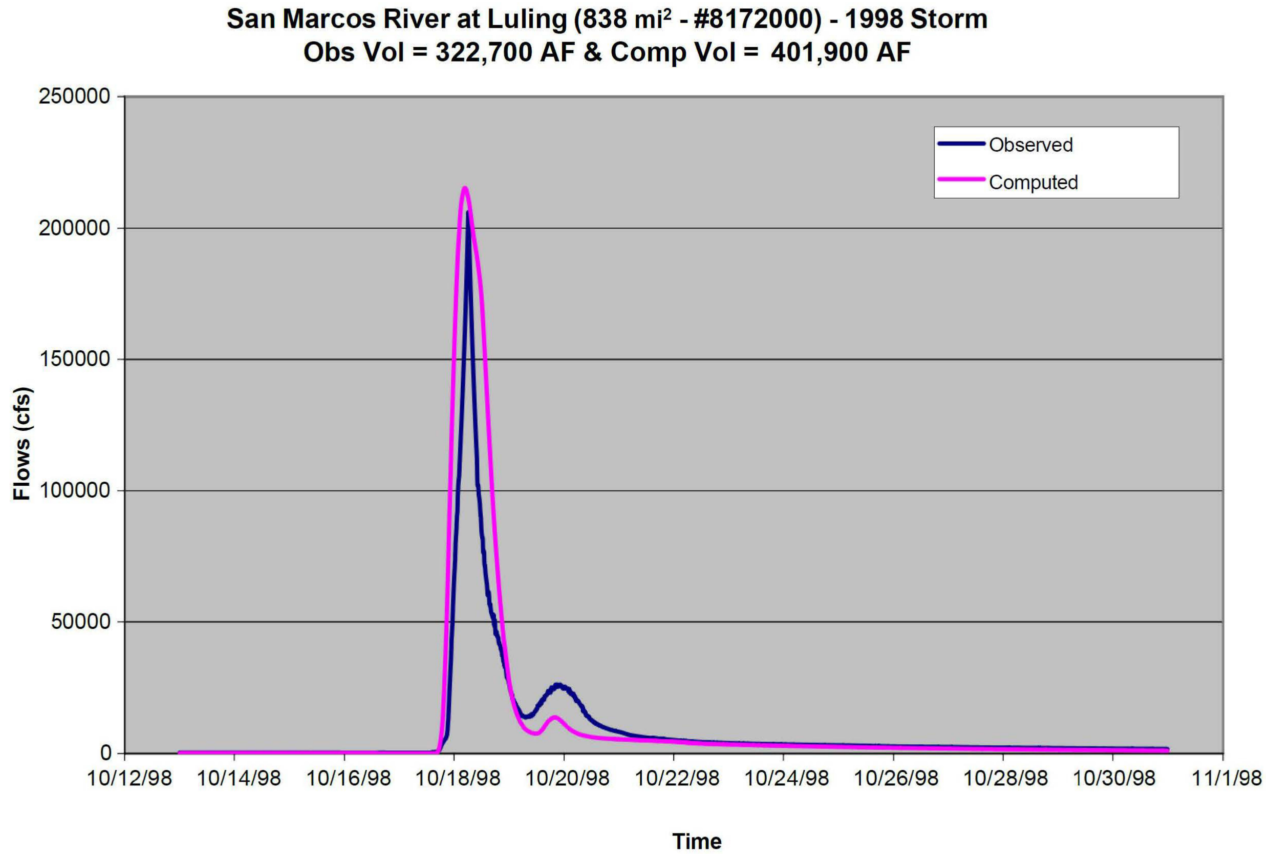


Figure 2.4.3-9
1998 Flood — Observed and Computed Hydrographs, Sandies Creek near Westhoff (USGS No. 8175000)

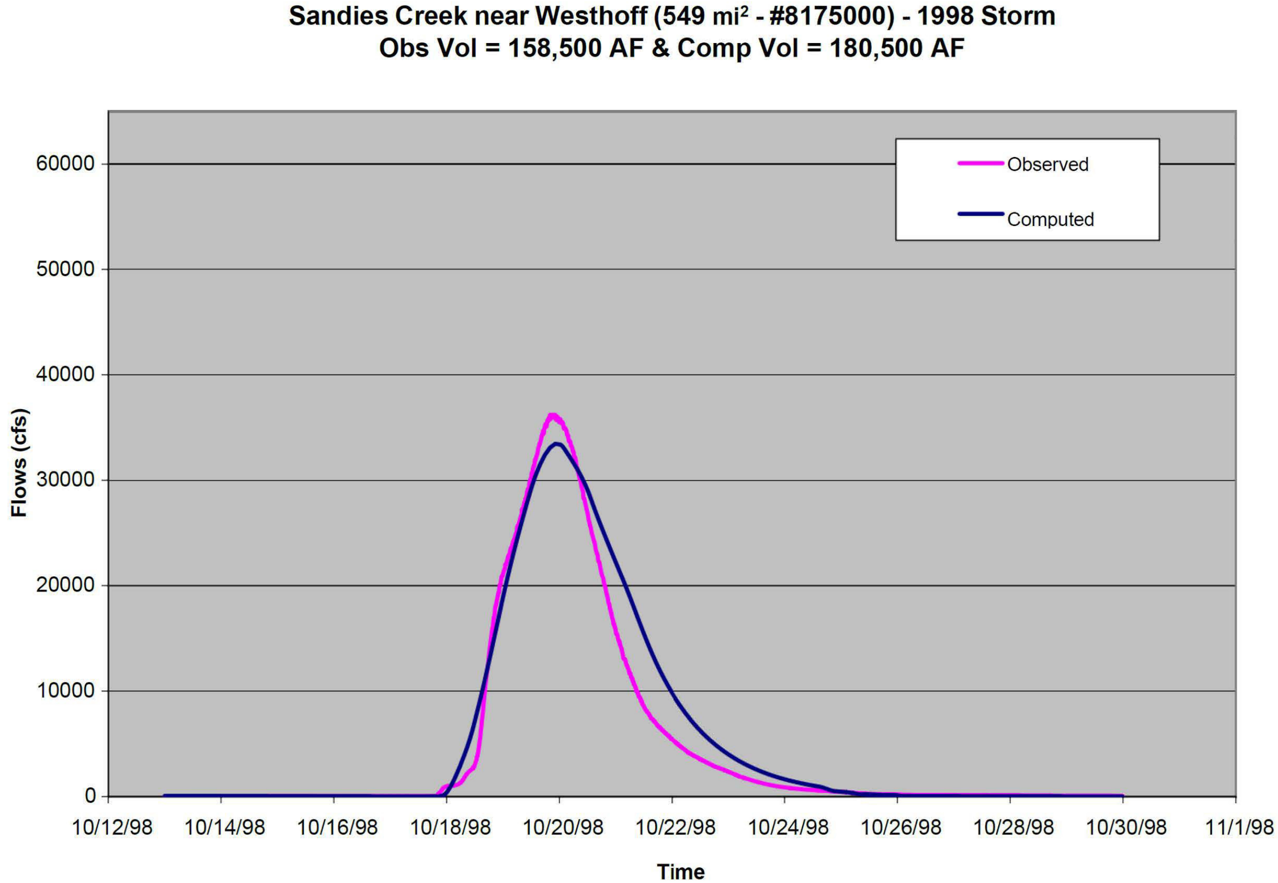


Figure 2.4.3-10
1998 Flood — Observed and Computed Hydrographs, Guadalupe River at Cureo (USGS No. 8175800)

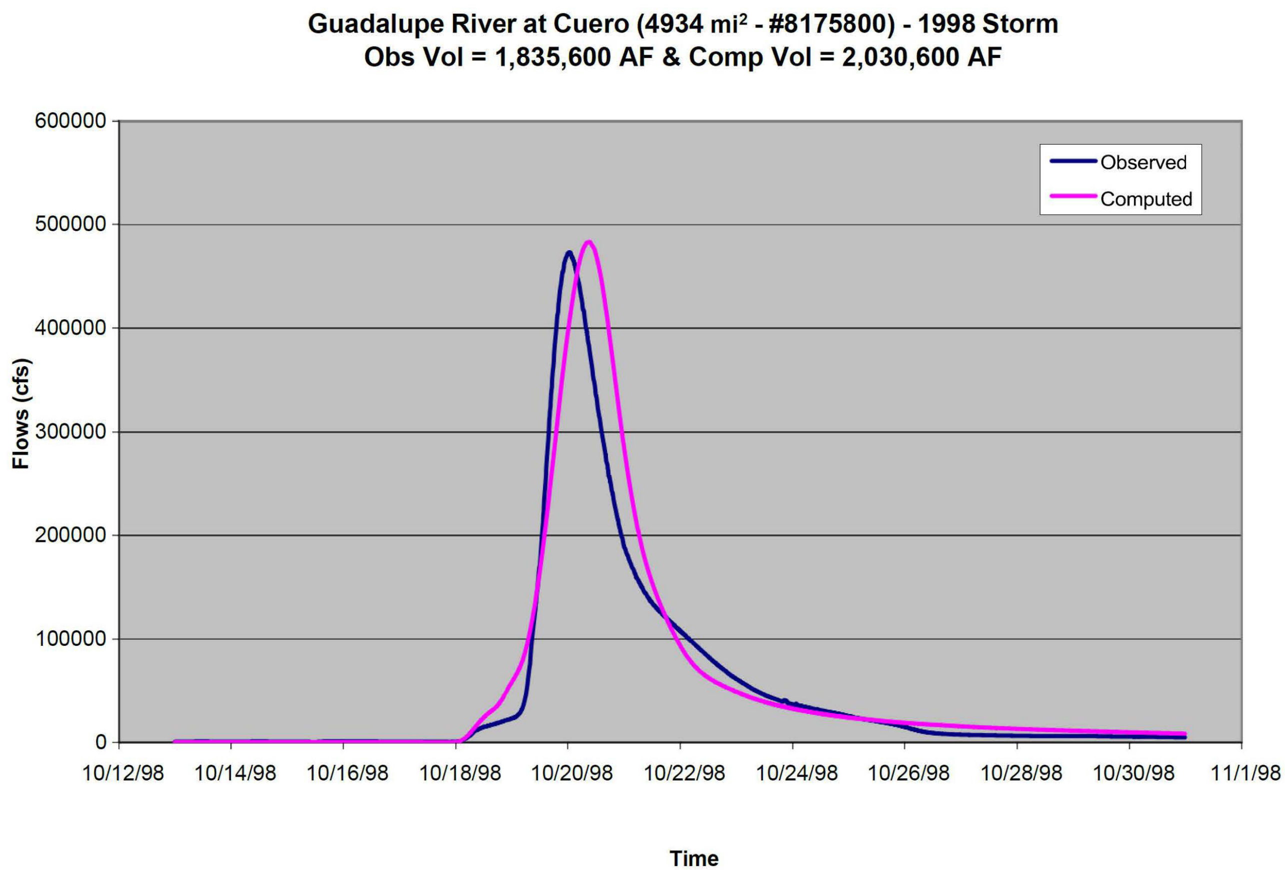


Figure 2.4.3-11
1998 Flood — Observed and Computed Hydrographs, Guadalupe River at Victoria (USGS no. 8176500)

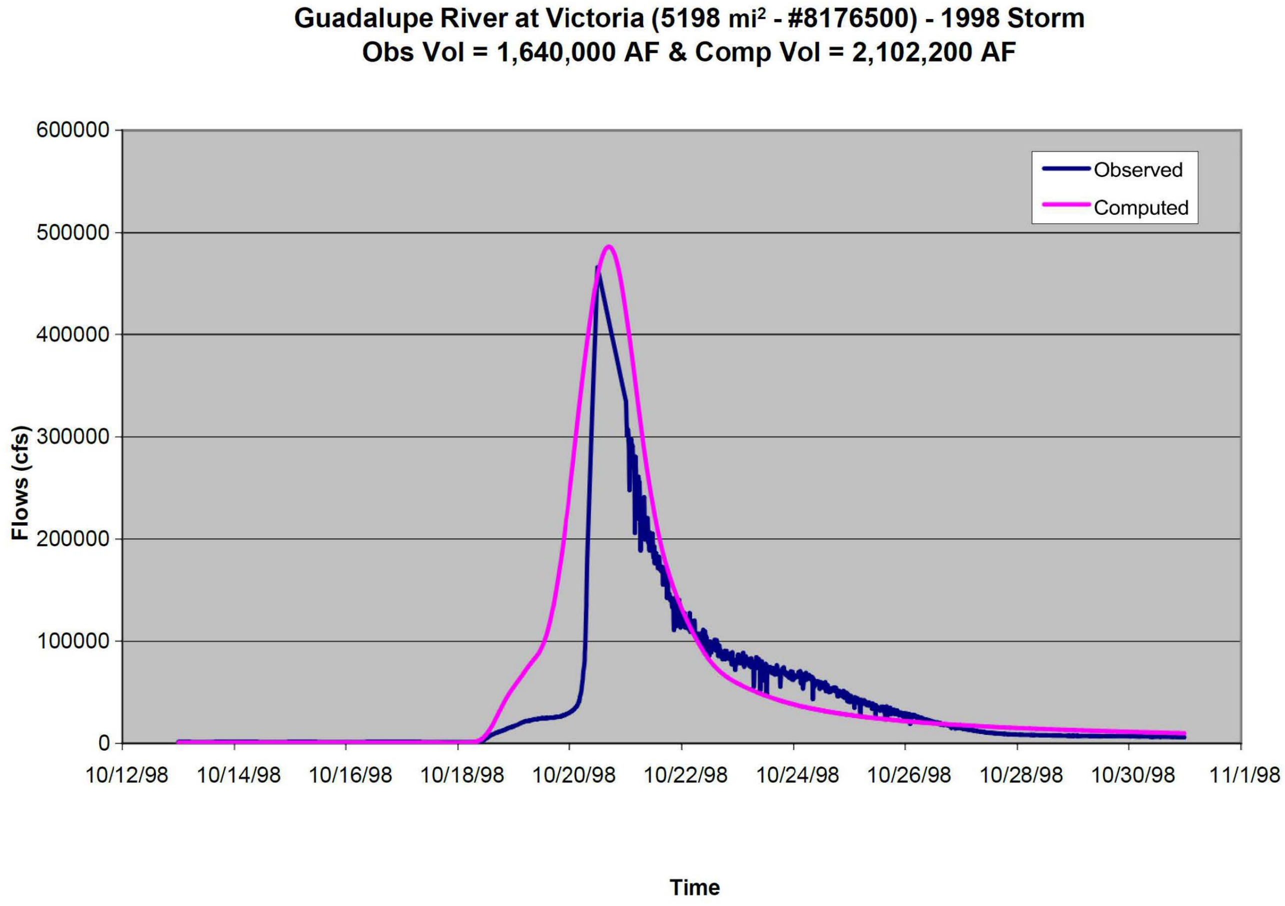


Figure 2.4.3-12
1998 Flood — Observed and Computed Hydrographs, Coleto Creek at Road Crossing near Schroeder (USGS No. 8176900)

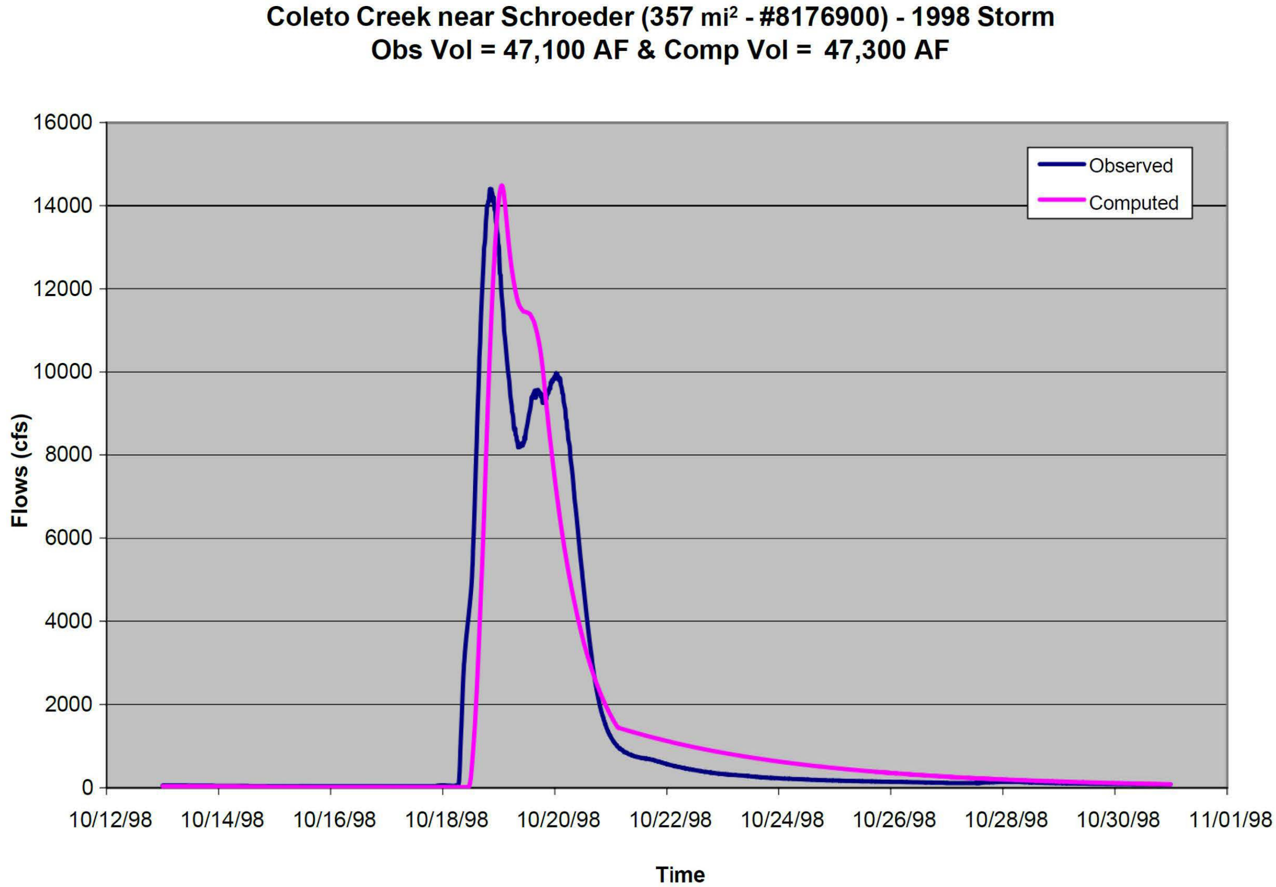


Figure 2.4.3-13
1998 Flood — Observed and Computed Hydrographs, Coleto Creek near Victoria (USGS No. 8177500)

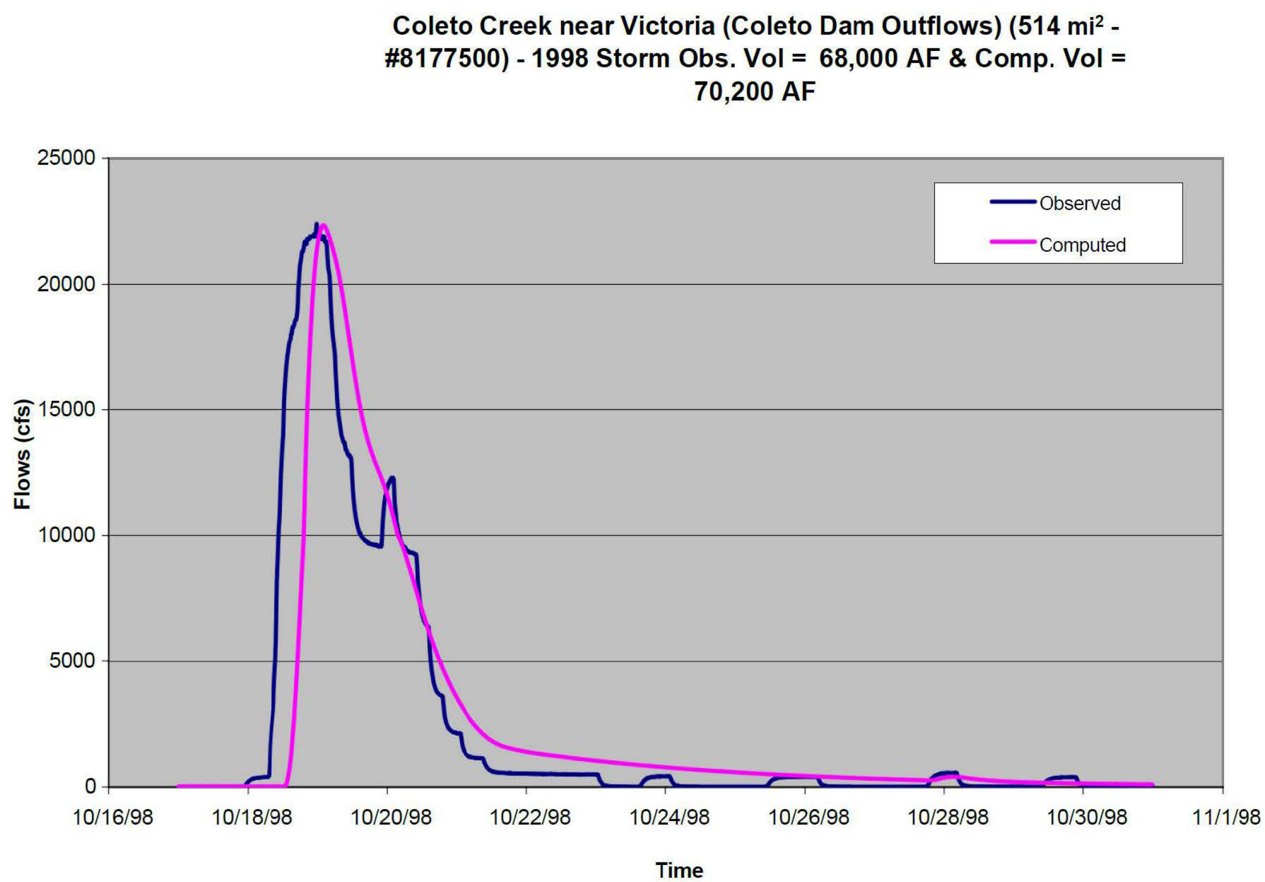


Figure 2.4.3-14
2004 Flood — Observed and Computed Hydrographs, Guadalupe River above Comal River (USGS No. 8168500)

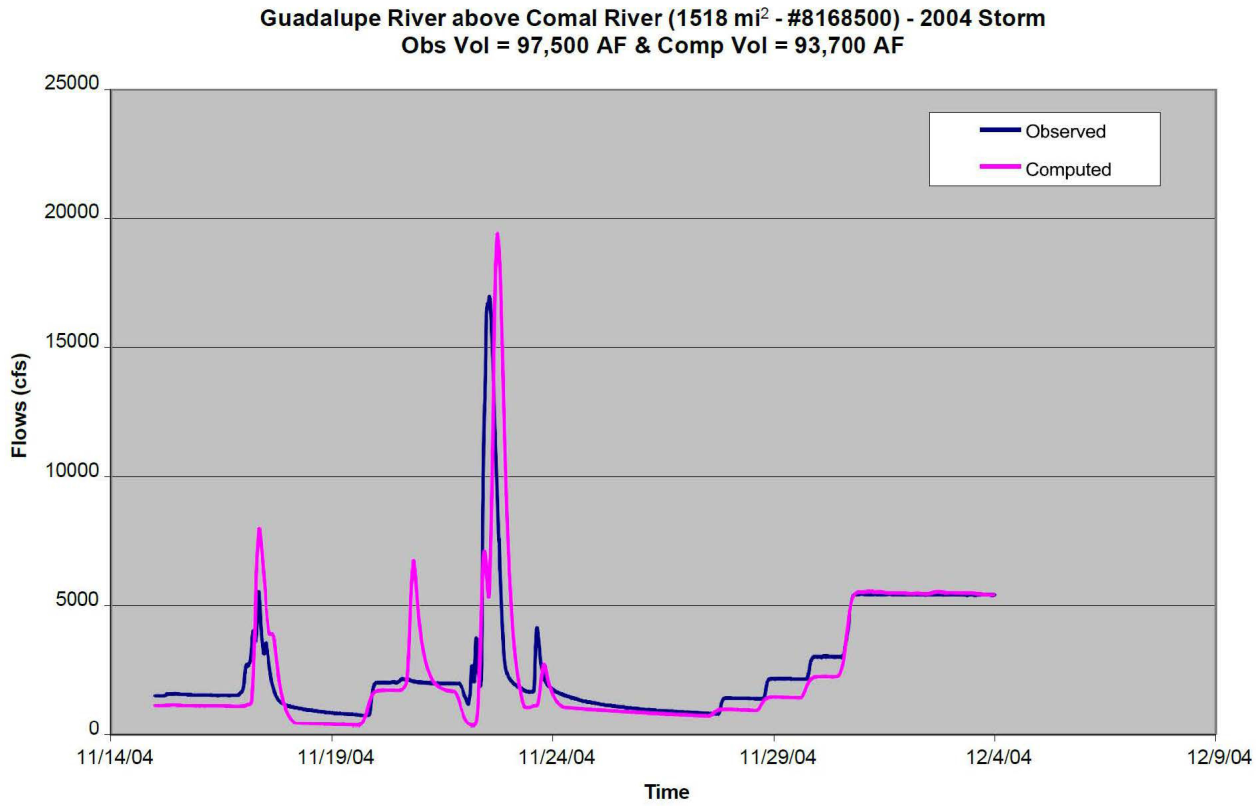


Figure 2.4.3-15
2004 Flood — Observed and Computed Hydrographs, Blanco River at Wimberley (USGS No. 8171000)

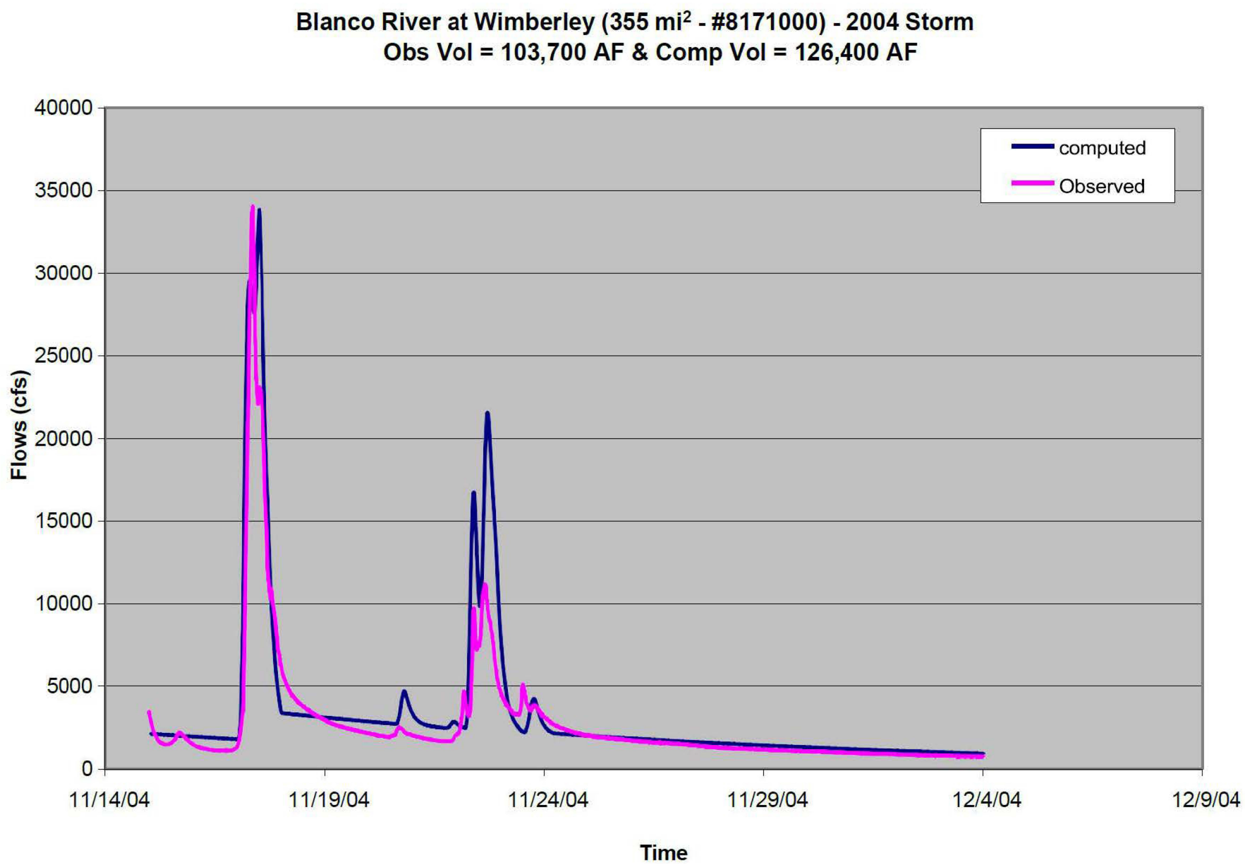


Figure 2.4.3-16
2004 Flood — Observed and Computed Hydrographs, Plum Creek at Lockhart (USGS No. 8172400)

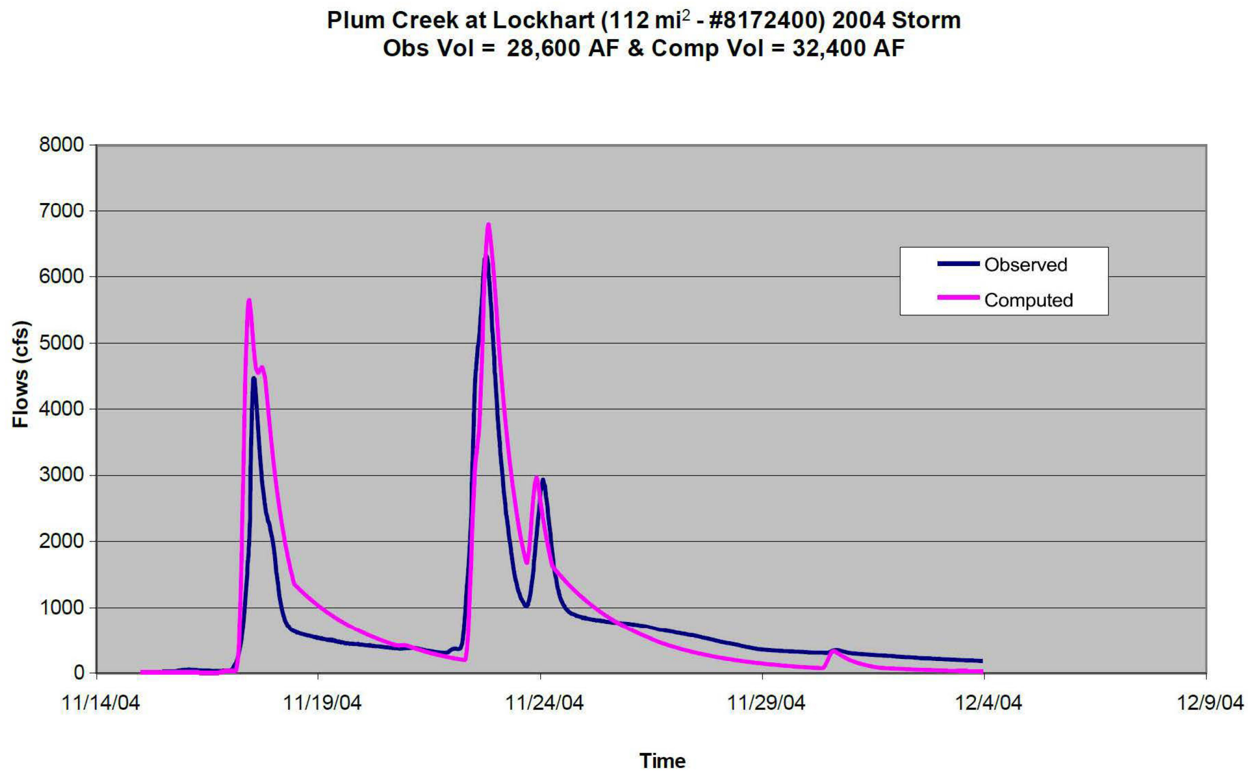


Figure 2.4.3-17
2004 Flood — Observed and Computed Hydrographs, Plum Creek near Luling (USGS No. 8173000)

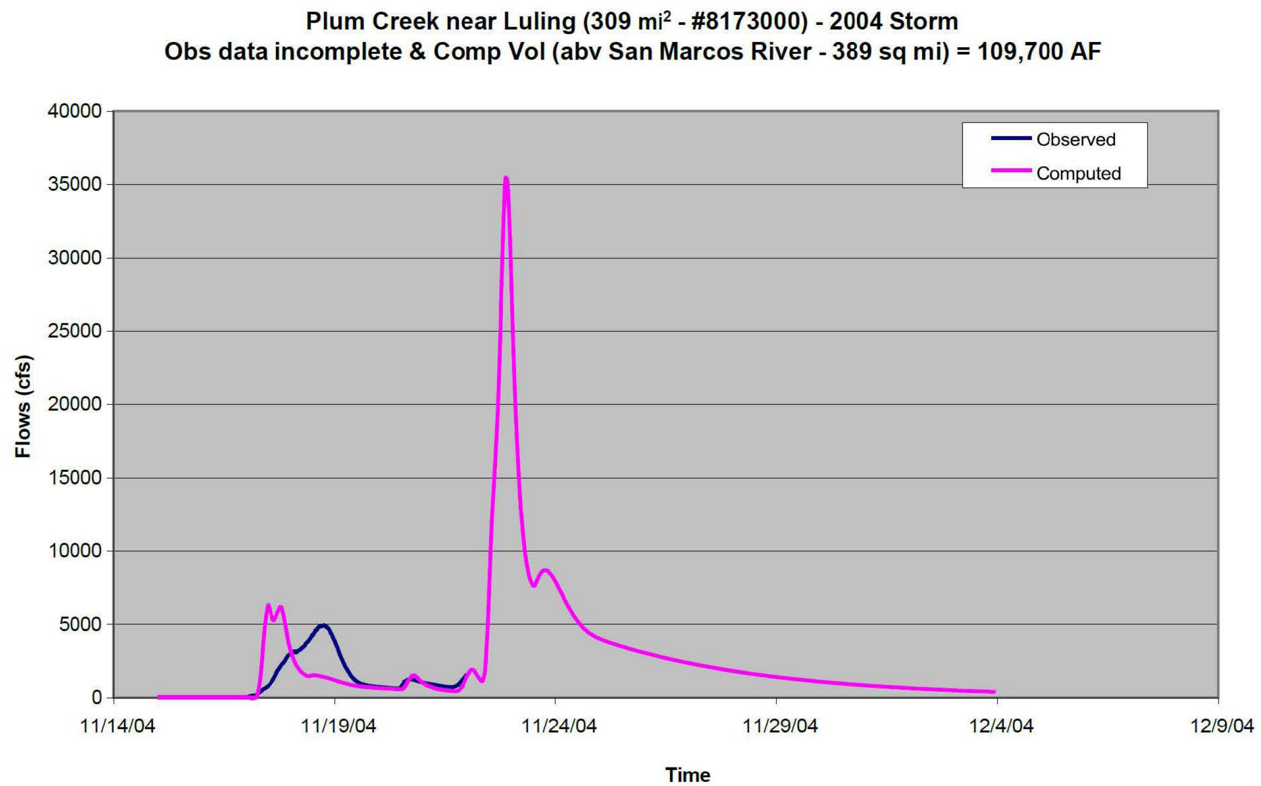


Figure 2.4.3-18
2004 Flood — Observed and Computed Hydrographs, San Marcos River at Luling (USGS No. 8172000)

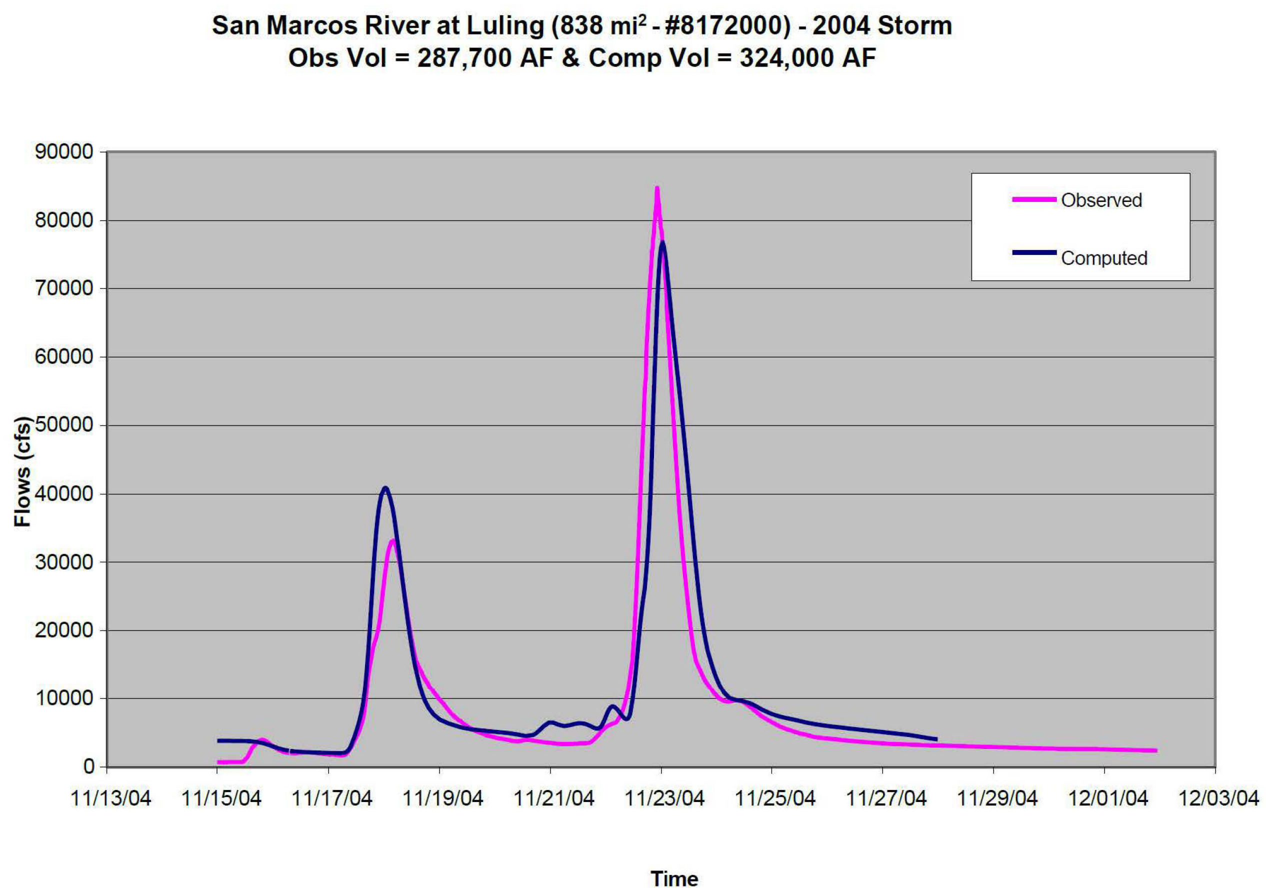


Figure 2.4.3-19
2004 Flood — Observed and Computed Hydrographs, Guadalupe River at Gonzales (USGS No. 8173900)

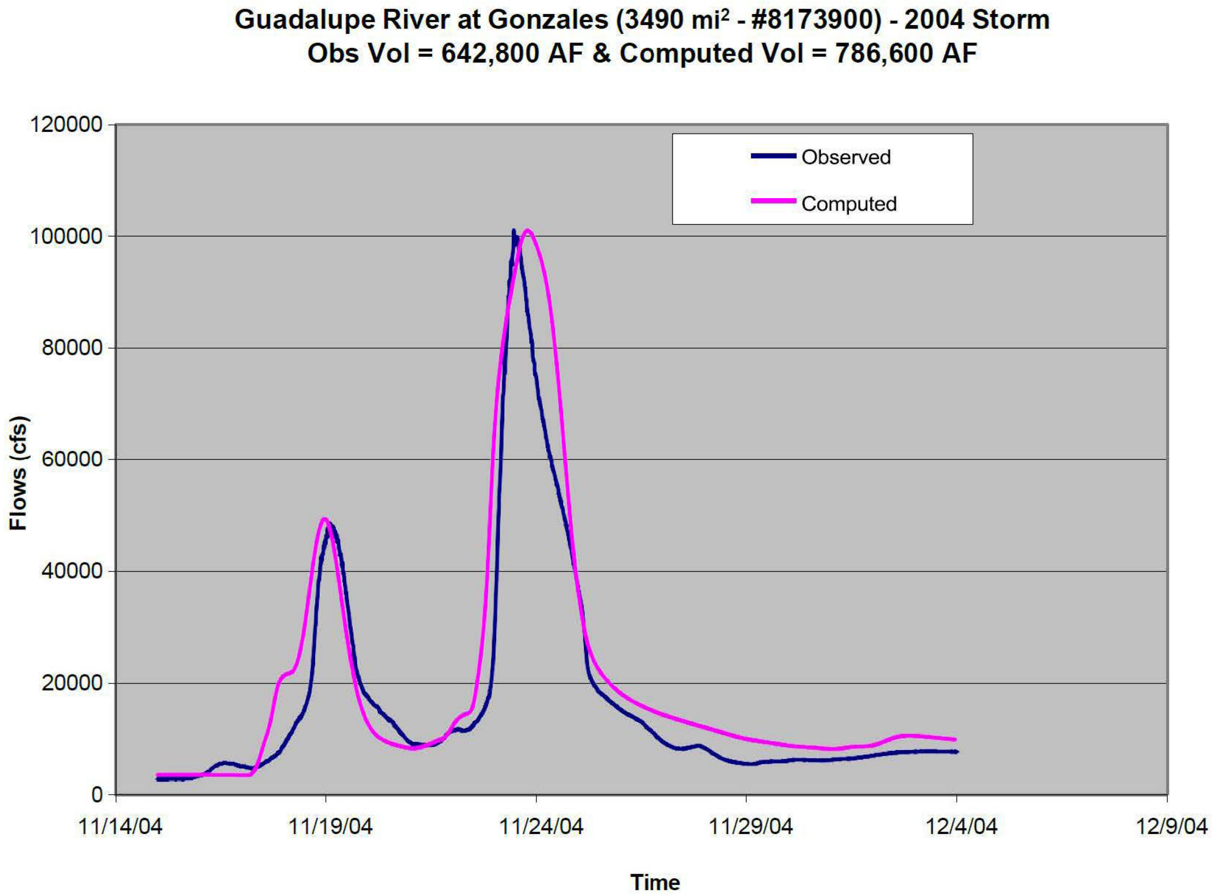


Figure 2.4.3-20
2004 Flood — Observed and Computed Hydrographs, Peach Creek at Dilworth (USGS No. 8174600)

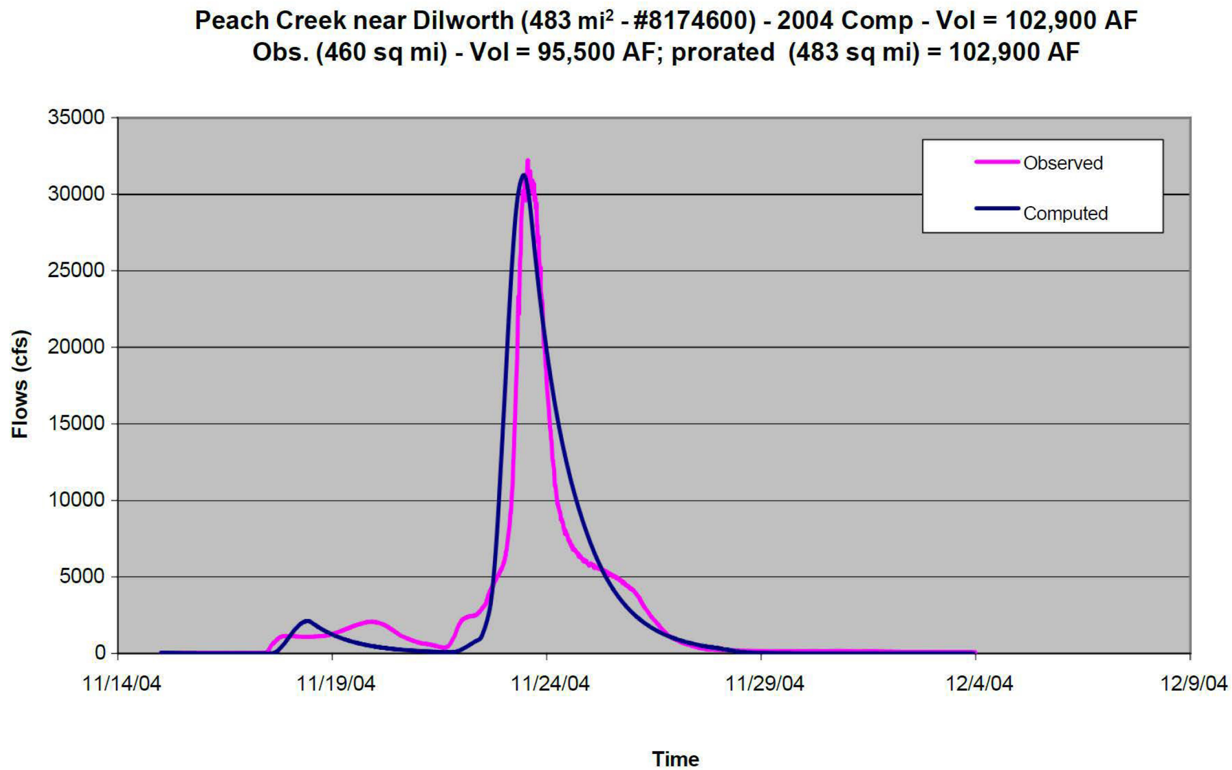


Figure 2.4.3-21
2004 Flood — Observed and Computed Hydrographs, Sandies Creek near Westhoff (USGS No. 8175000)

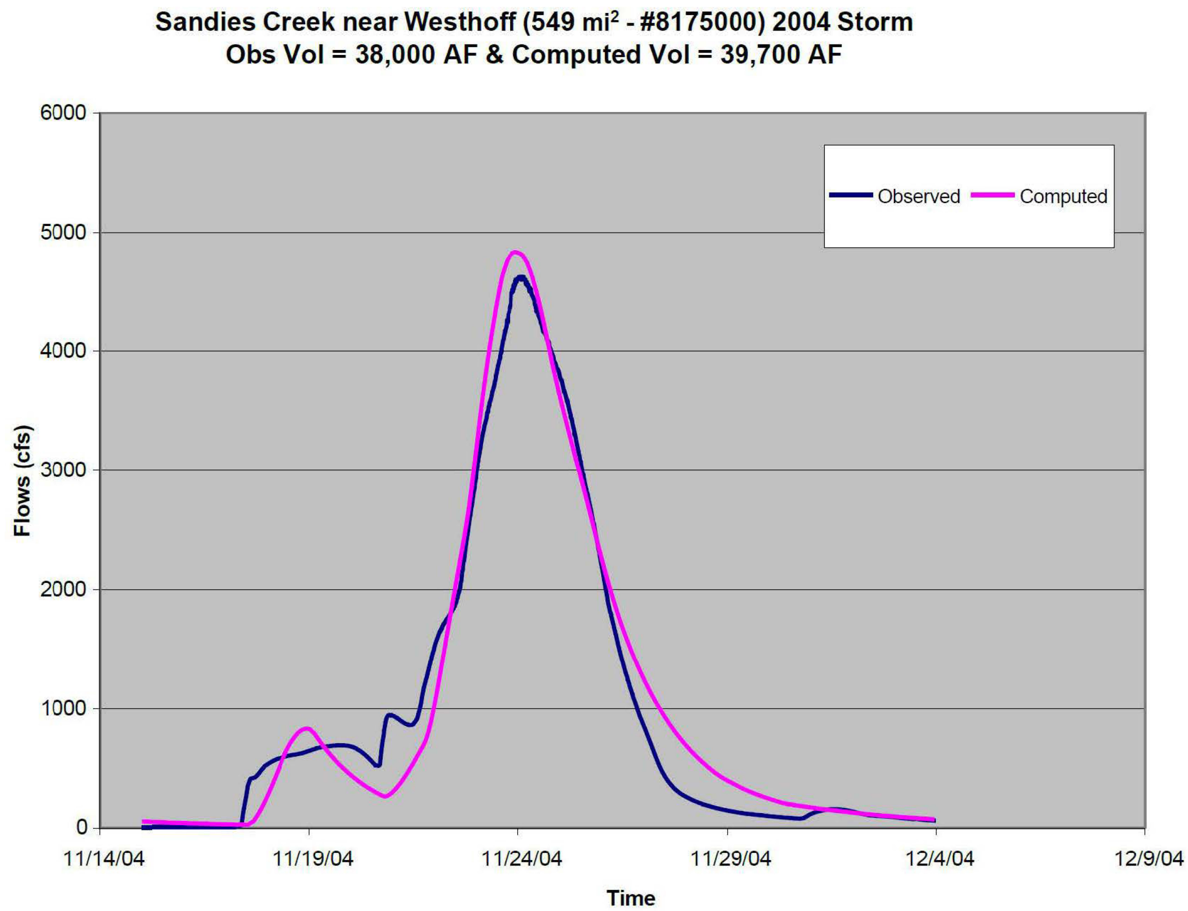


Figure 2.4.3-22
2004 Flood — Observed and Computed Hydrographs, Guadalupe River at Cuero (USGS No. 8175800)

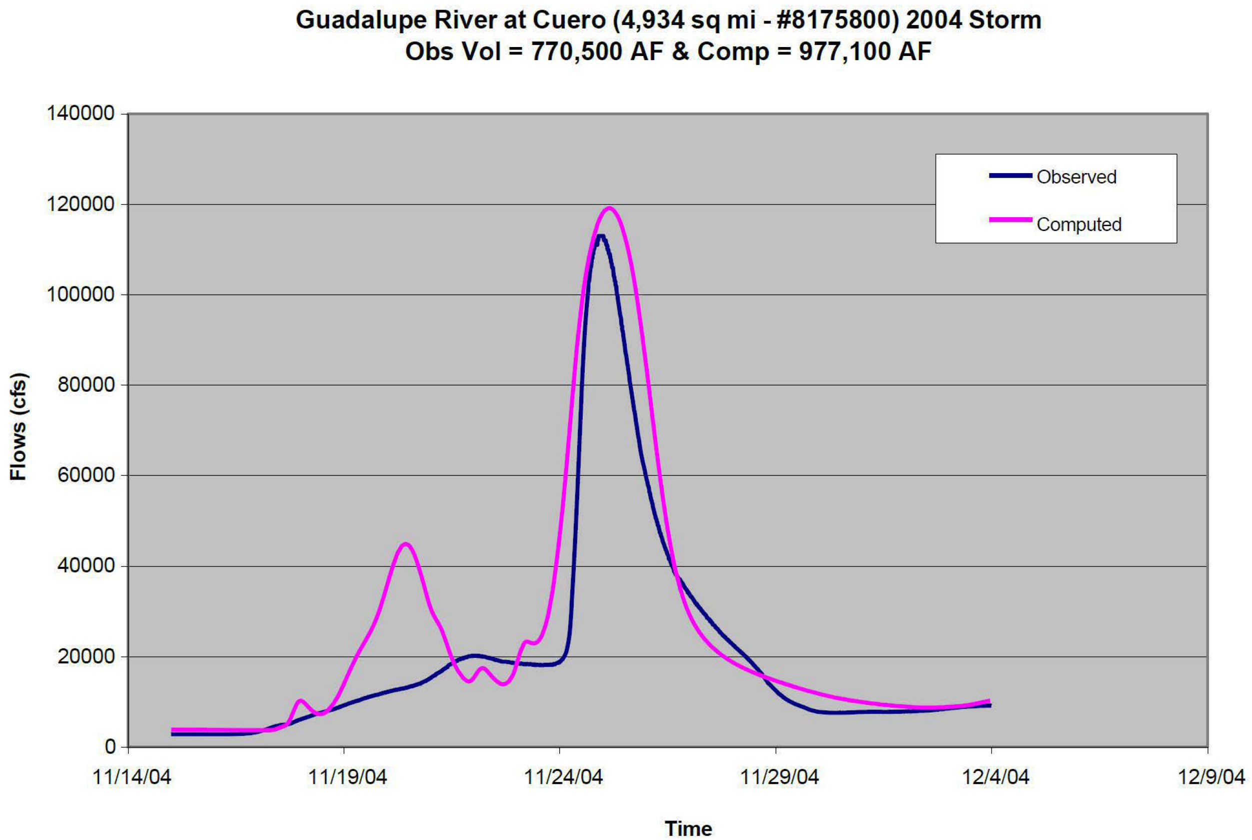


Figure 2.4.3-23
2004 Flood — Observed and Computed Hydrographs, Guadalupe River at Victoria (USGS No. 8176500)

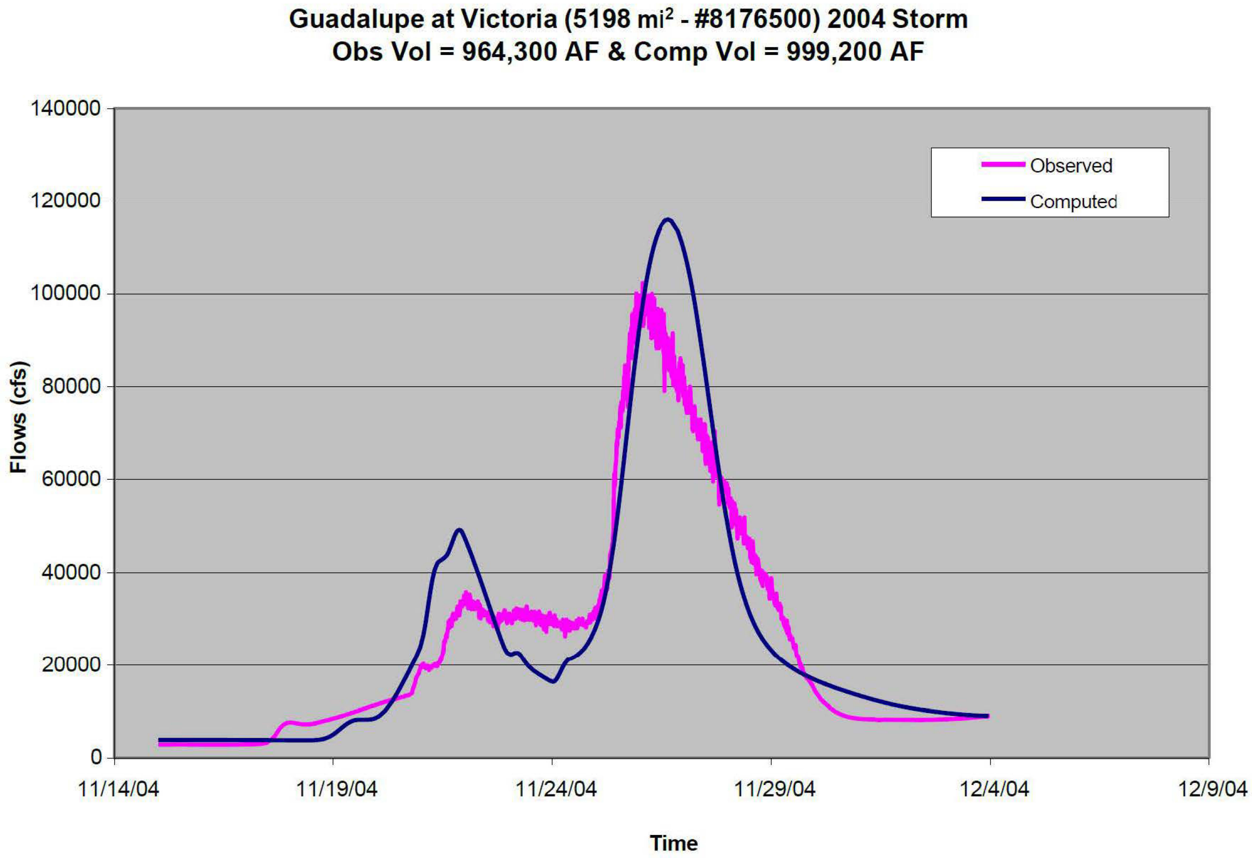


Figure 2.4.3-24
2004 Flood — Observed and Computed Hydrographs, Coleto Creek at Road Crossing near Schroeder (USGS No. 8176900)

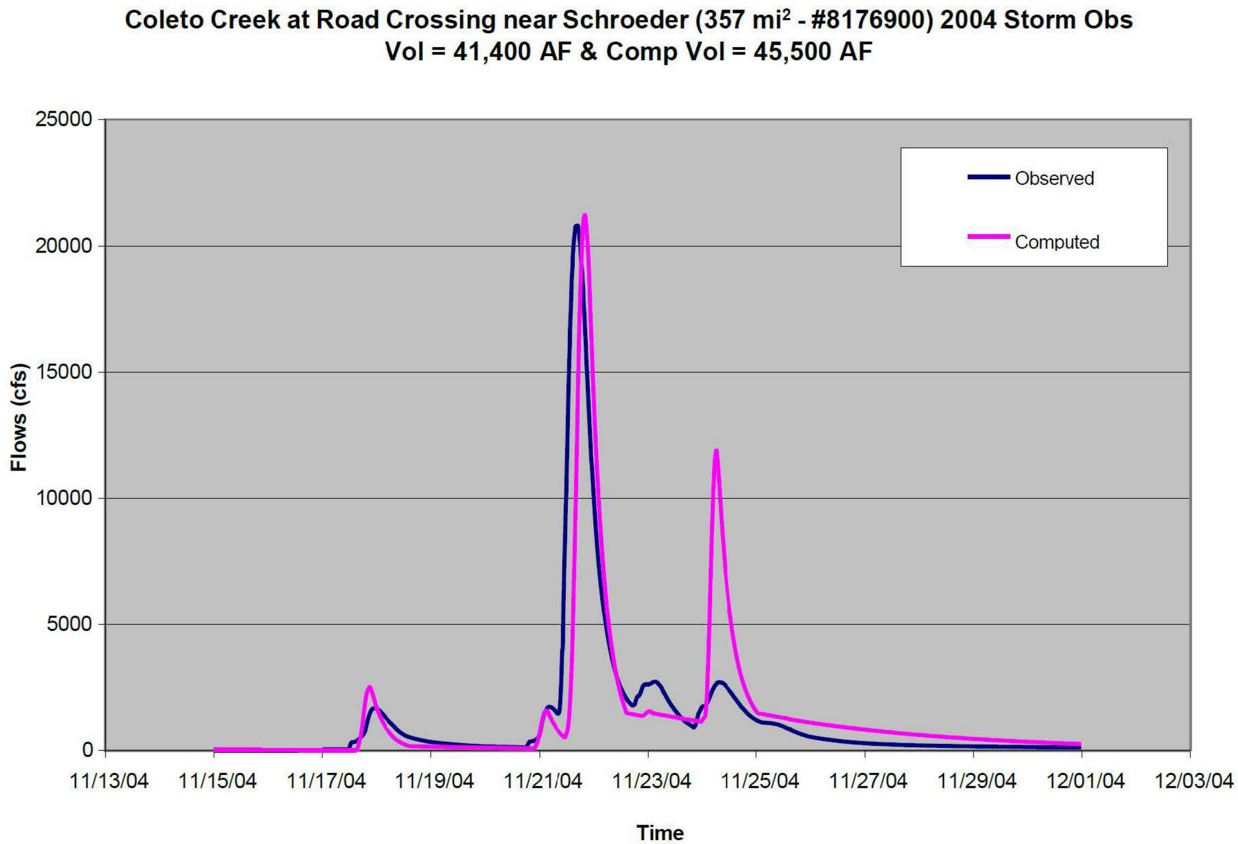


Figure 2.4.3-25
2004 Flood — Observed and Computed Hydrographs, Coleto Creek near Victoria (USGS No. 8176500)

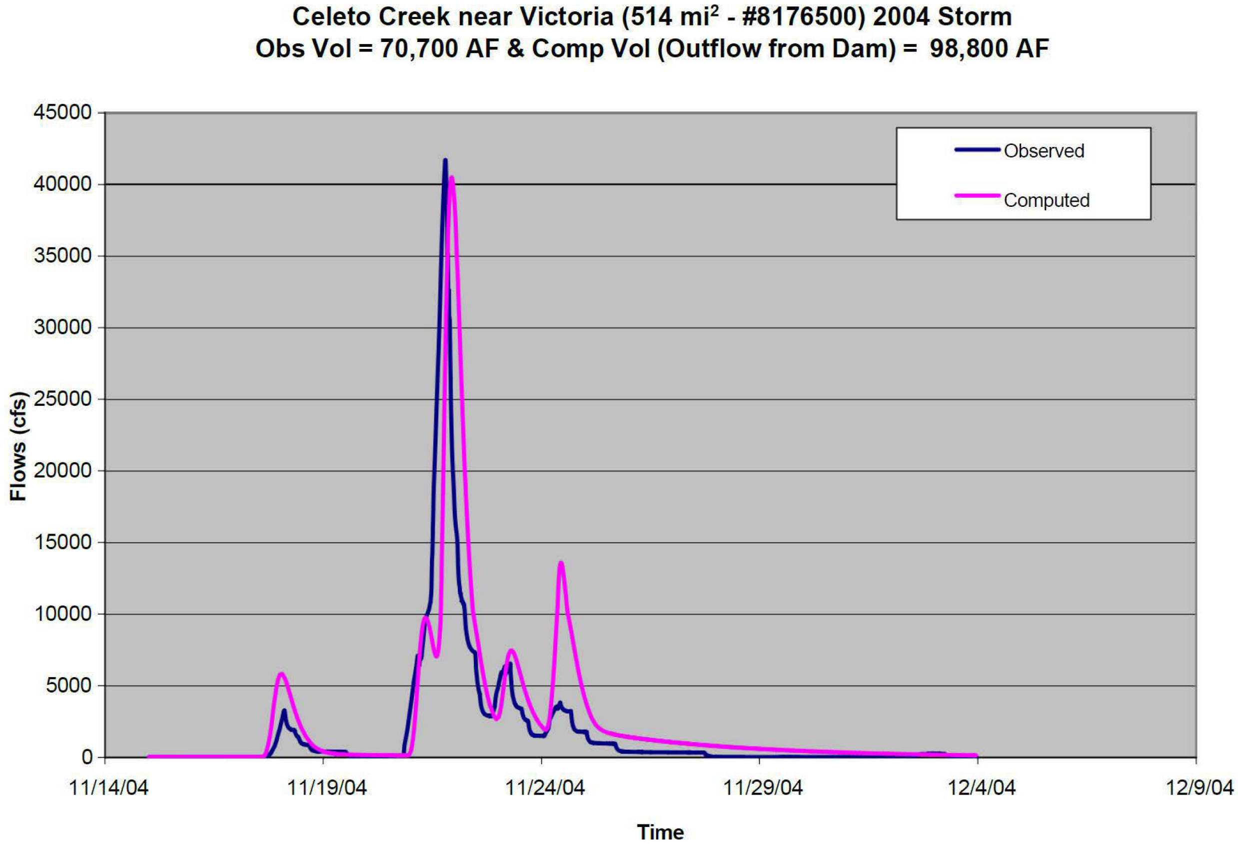
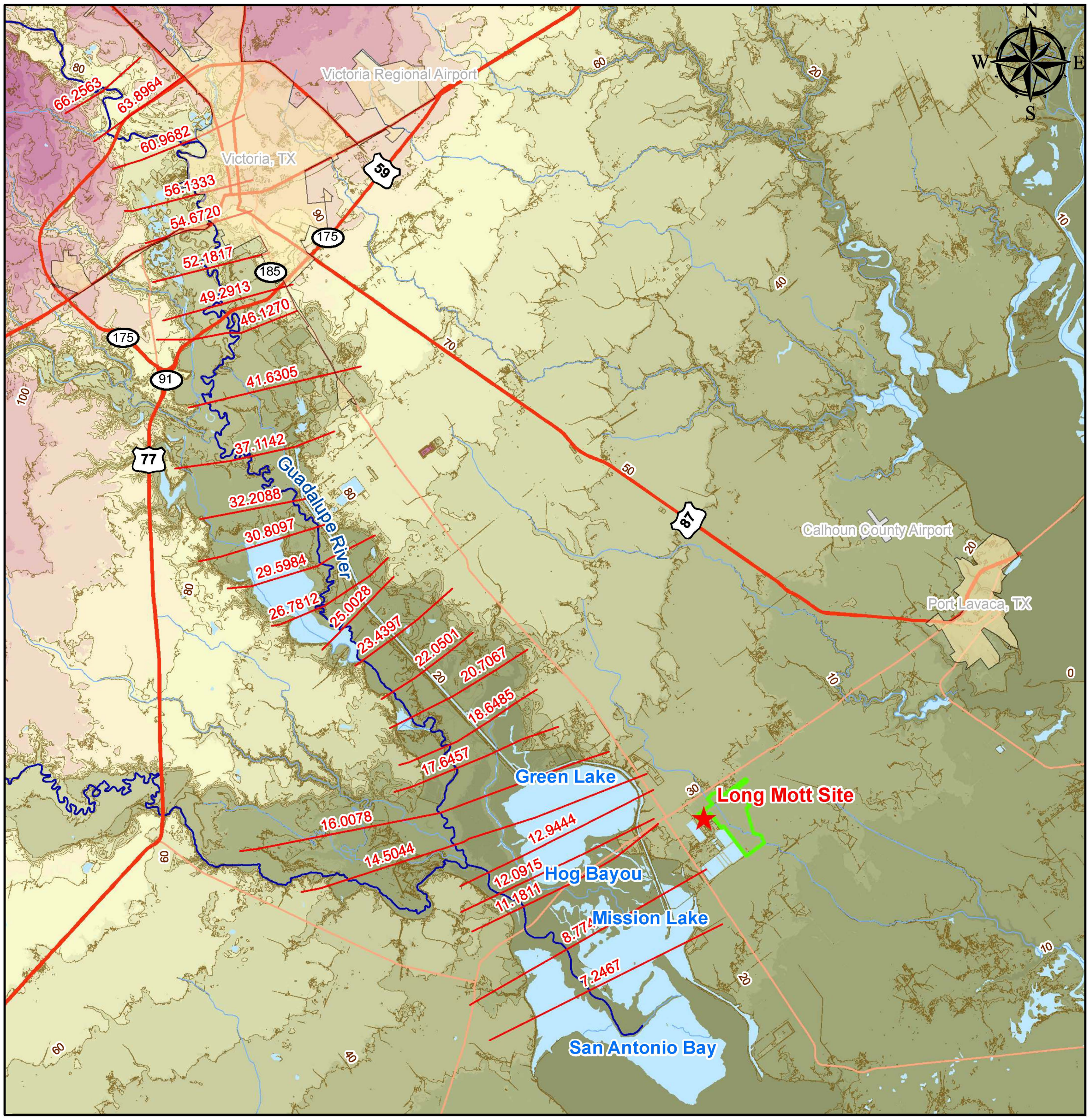
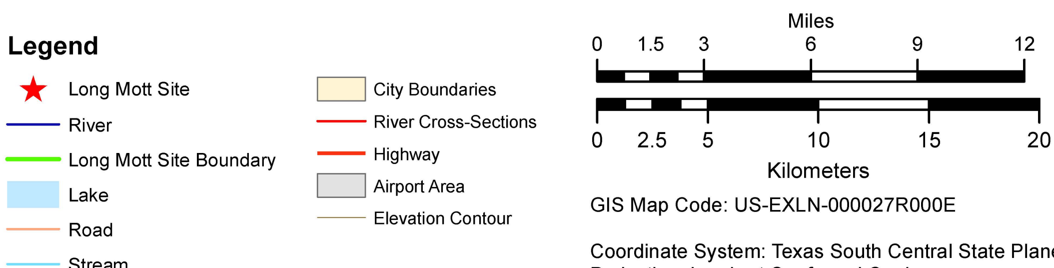


Figure 2.4.3-26
HEC-RAS Cross Section Locations





GIS Map Code: US-EXLN-000027R000E

Coordinate System: Texas South Central State Plane, FIPS 4204
Projection: Lambert Conformal Conic
Horizontal Datum: North American Datum 1983

Figure 2.4.3-27
HUC-12 Subwatershed Boundaries Adjacent to the Long Mott Generating Station Site

Figure 2.4.3-28
Tributary Areas to the Long Mott Generating Station Site

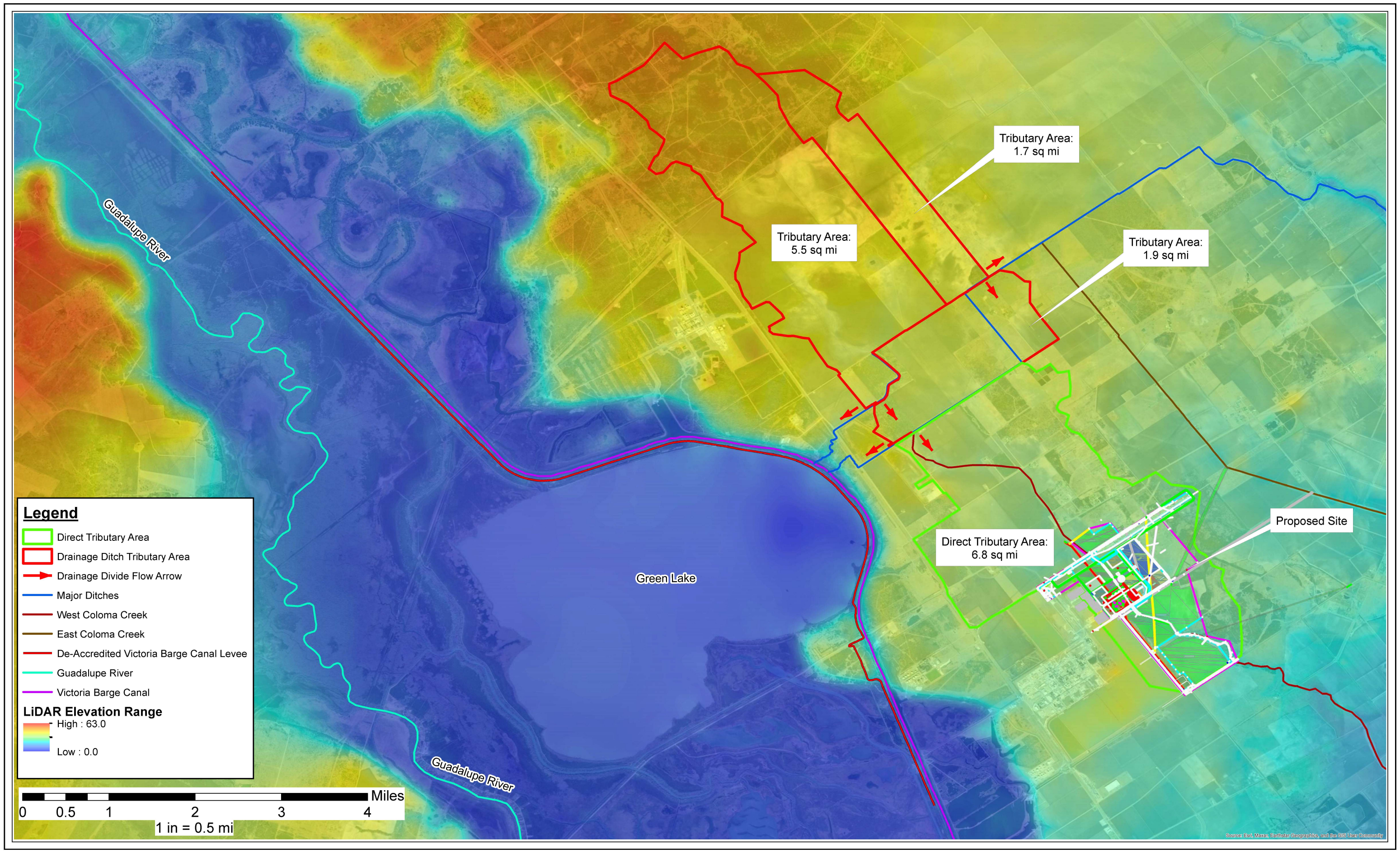


Figure 2.4.3-29
Direct Watershed Upstream of the Long Mott Generating Station Site

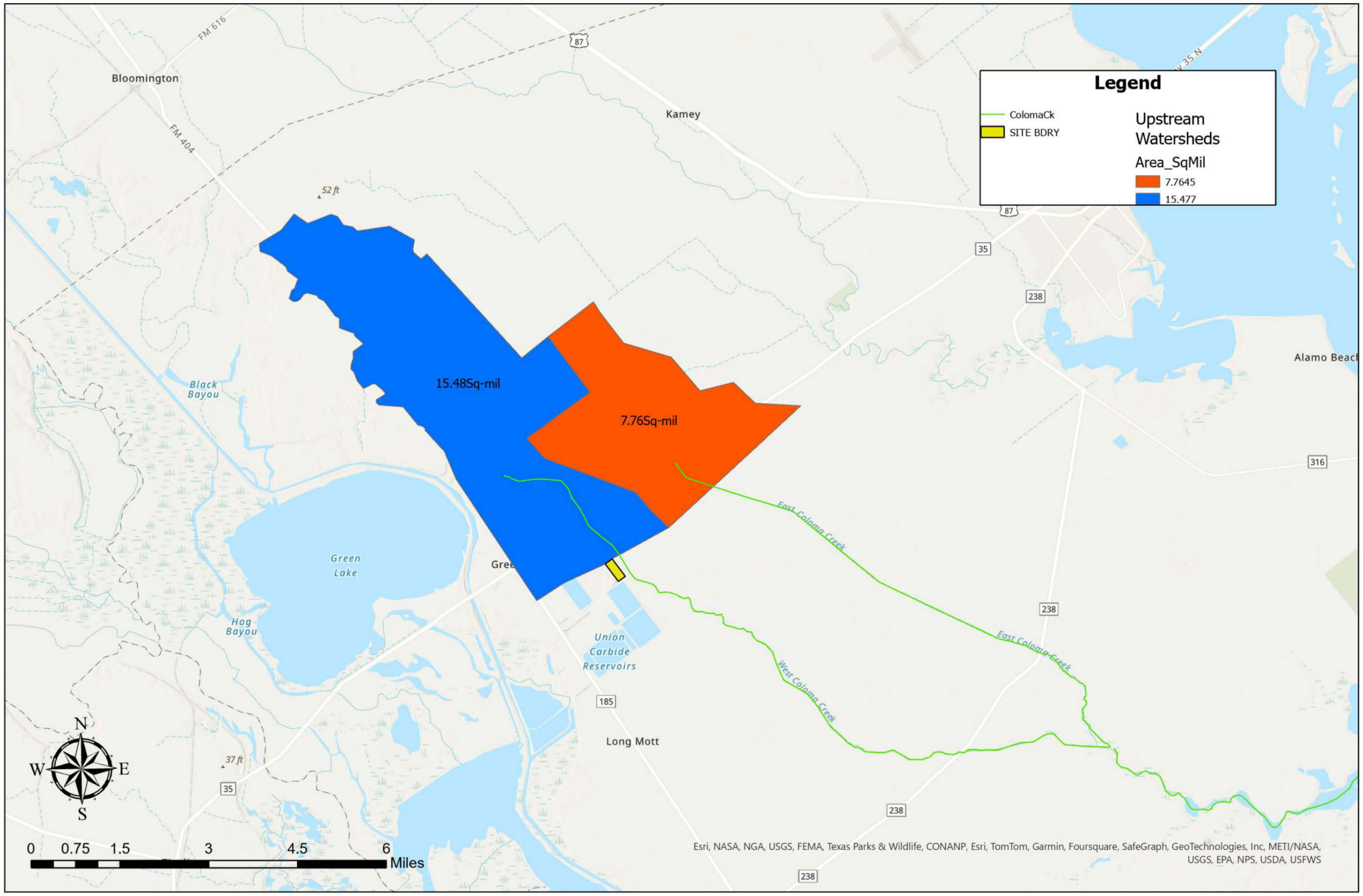


Figure 2.4.3-30
Sparks Watersheds, Hydraulic Structures, and Flow Direction

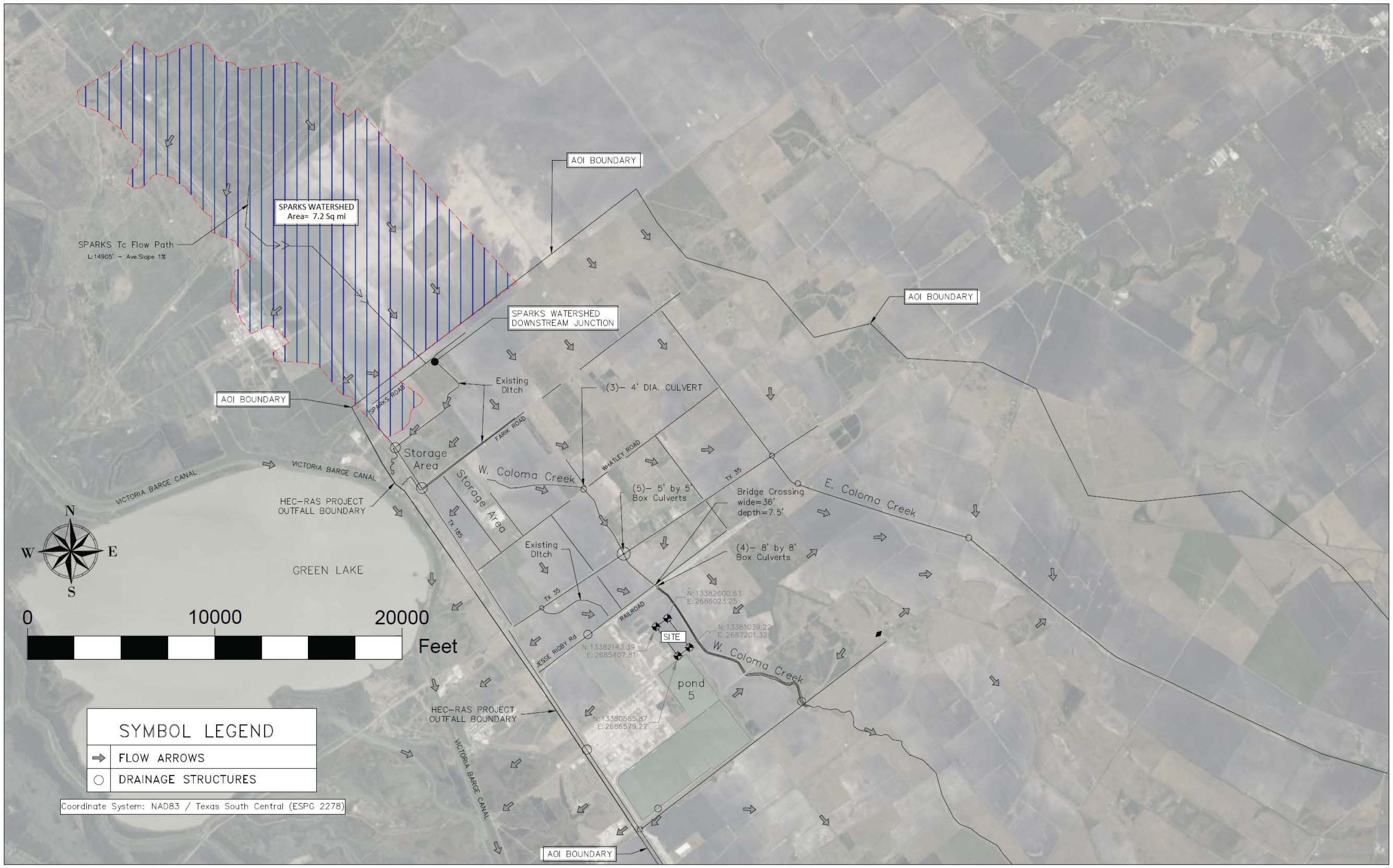


Figure 2.4.3-31
Drainage Ditches North of Site Tributary Area

Figure 2.4.3-32
Drainage Ditches North of Site Tributary Area – Blowup

Figure 2.4.3-33
Topography of West Coloma Creek Watershed below Sparks Road

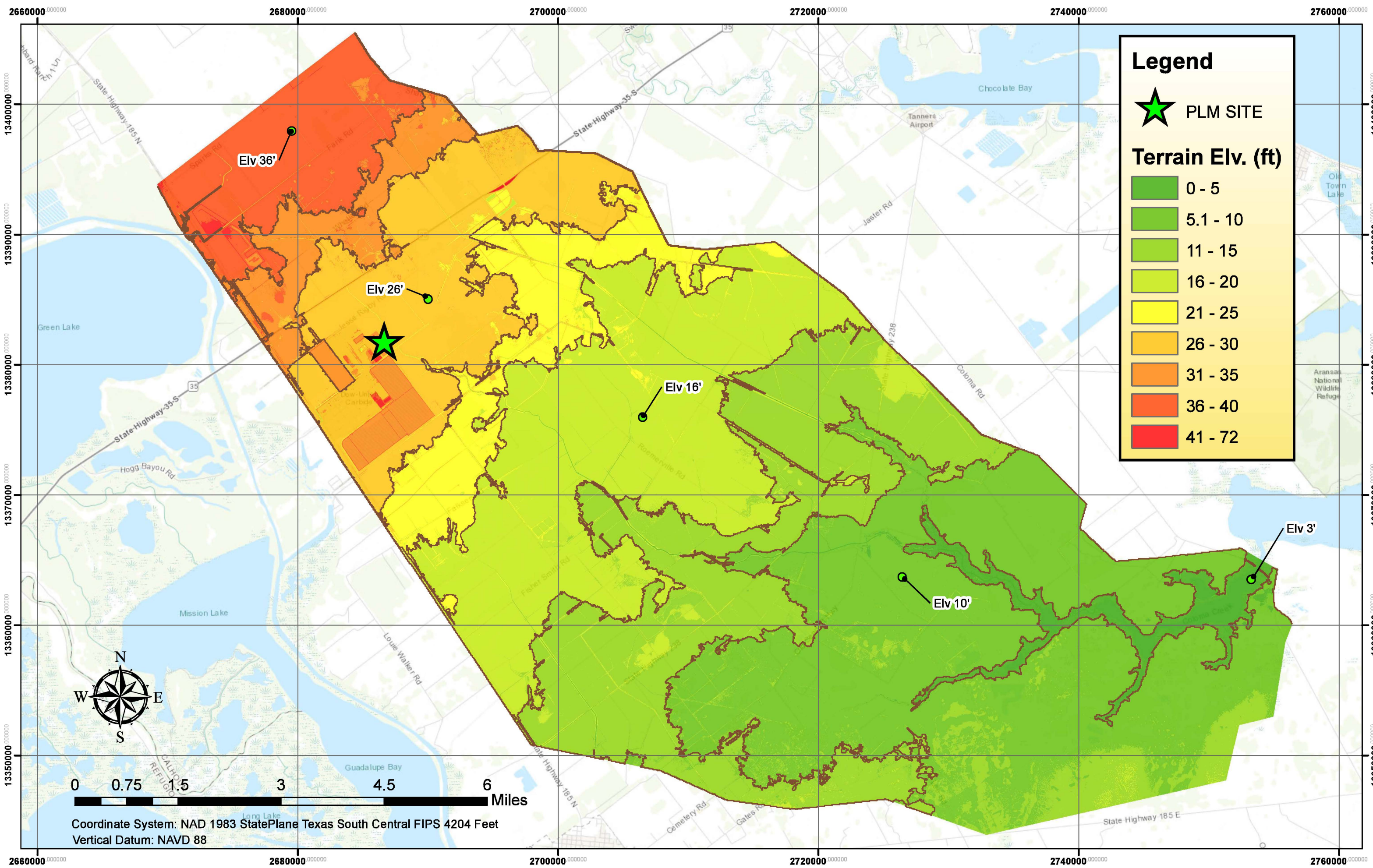


Figure 2.4.3-34
PMP Hyetograph for PMF over West Coloma Creek

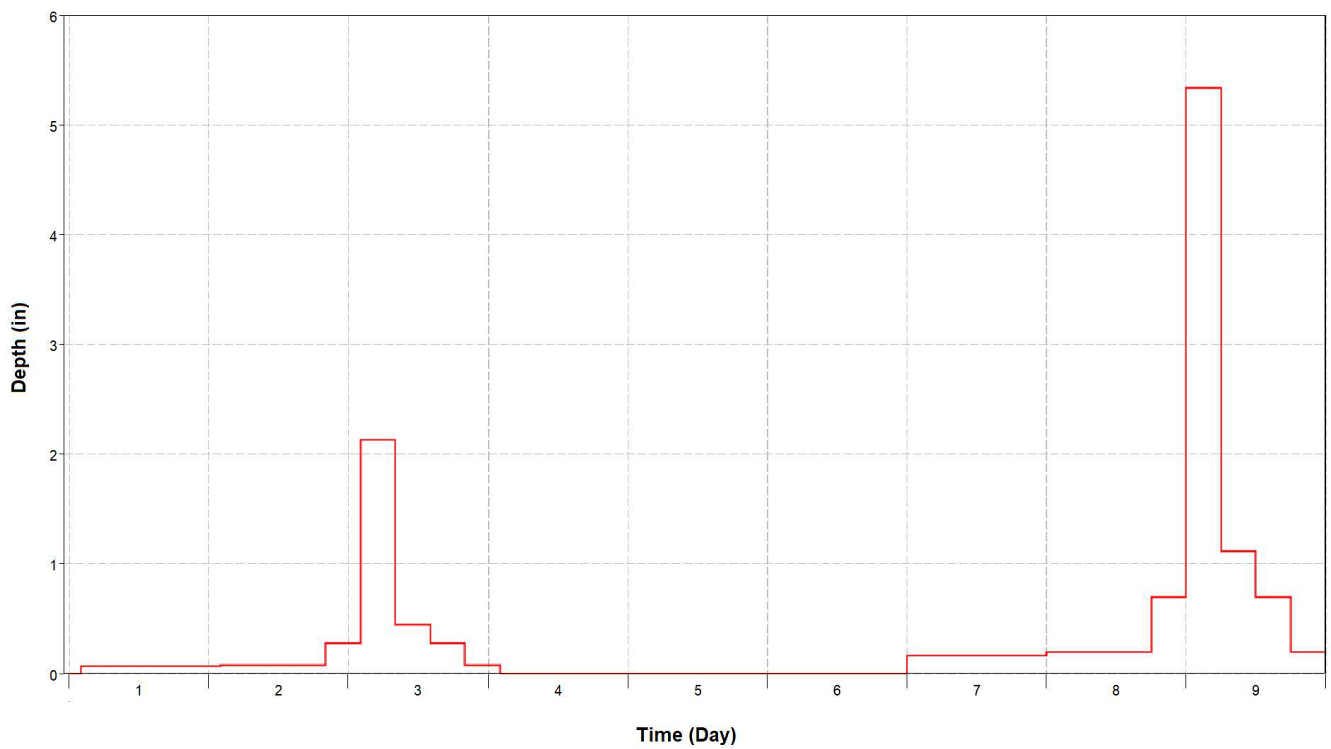
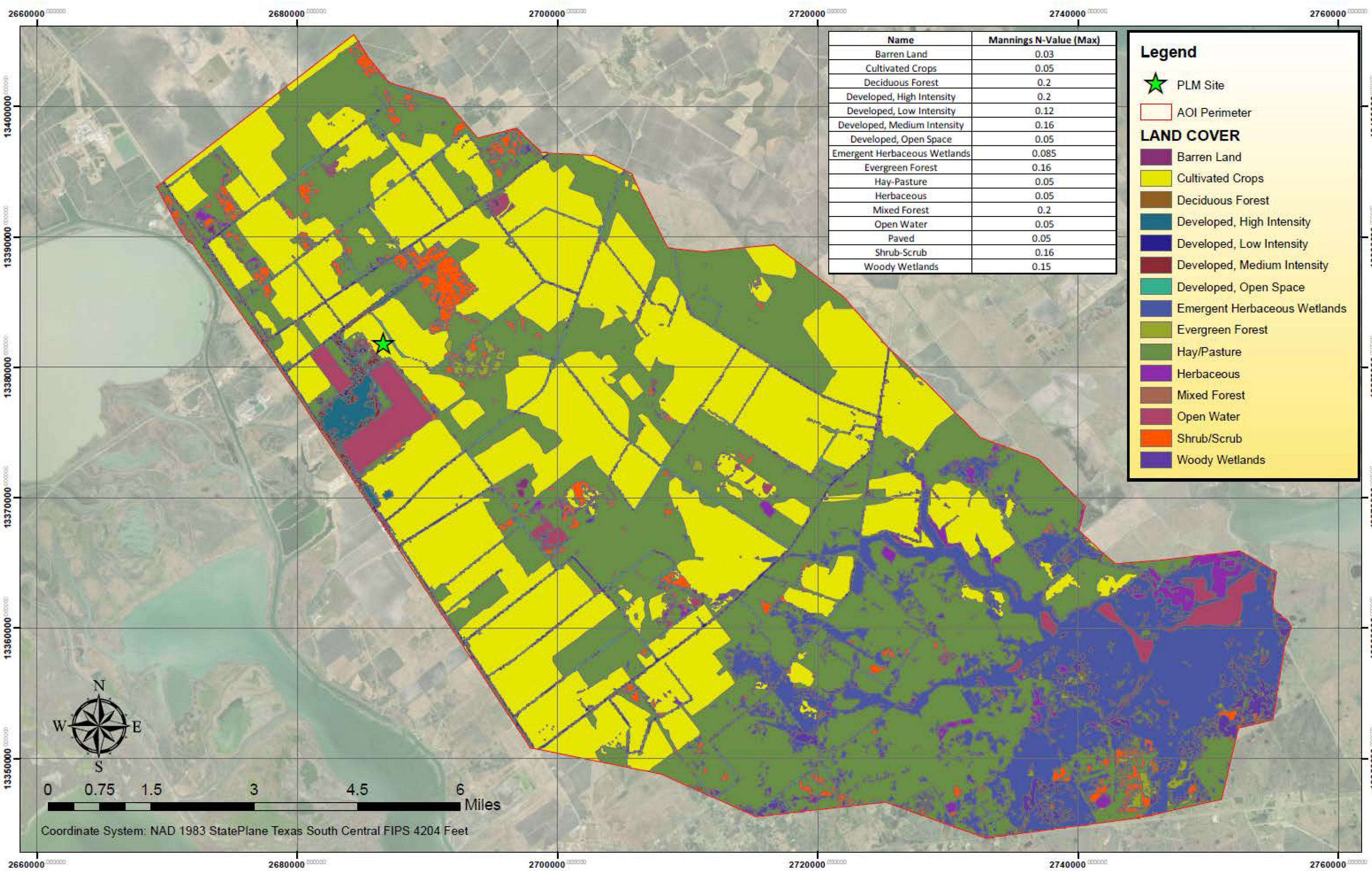


Figure 2.4.3-35
Watershed Land Use



Name	Mannings N-Value (Max)
Barren Land	0.03
Cultivated Crops	0.05
Deciduous Forest	0.2
Developed, High Intensity	0.2
Developed, Low Intensity	0.12
Developed, Medium Intensity	0.16
Developed, Open Space	0.05
Emergent Herbaceous Wetlands	0.085
Evergreen Forest	0.16
Hay-Pasture	0.05
Herbaceous	0.05
Mixed Forest	0.2
Open Water	0.05
Paved	0.05
Shrub-Scrub	0.16
Woody Wetlands	0.15

Figure 2.4.3-36
Watershed Hydrologic Soil Group

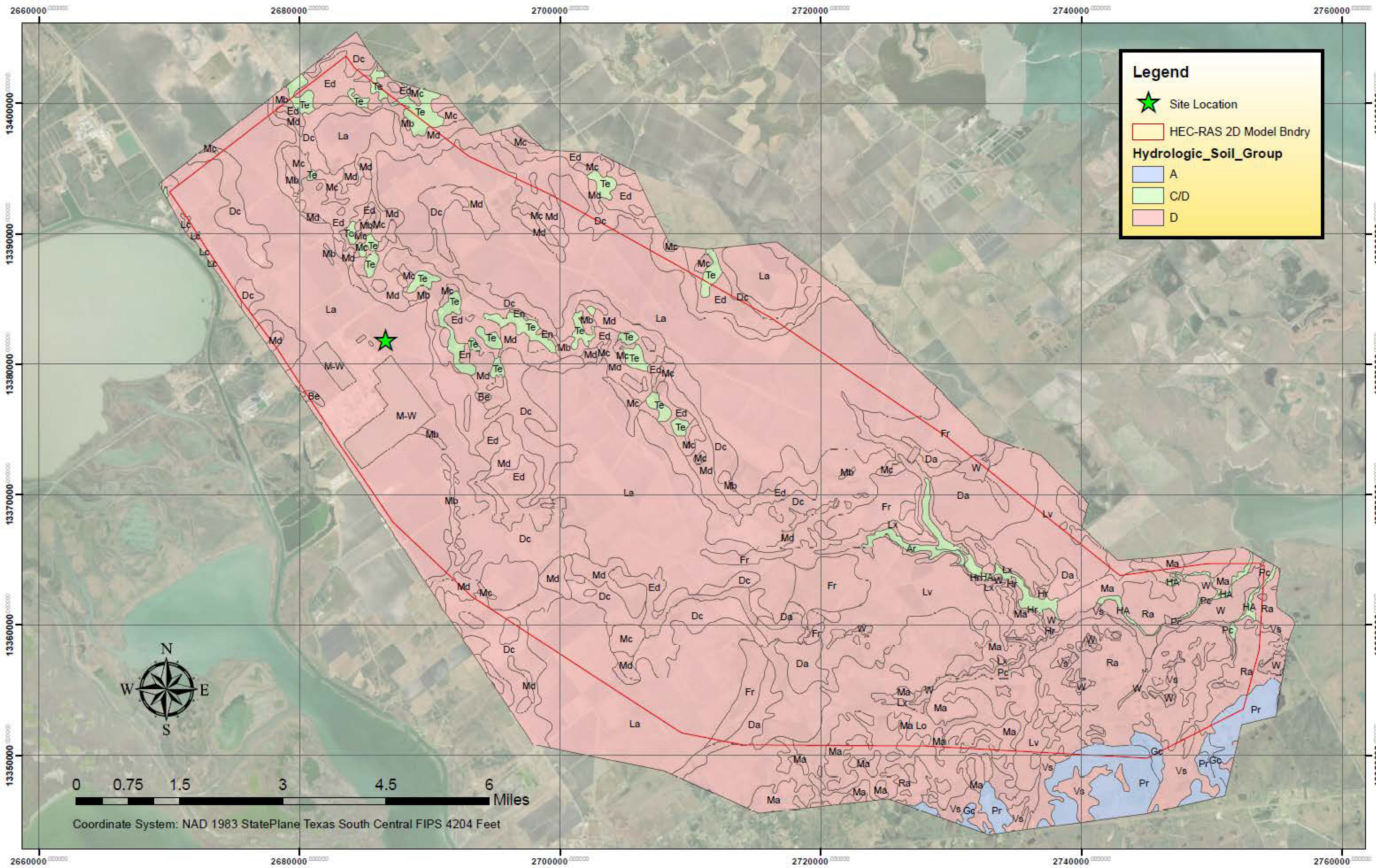


Figure 2.4.3-37
HEC-RAS 2D Model Outline and Boundary Conditions

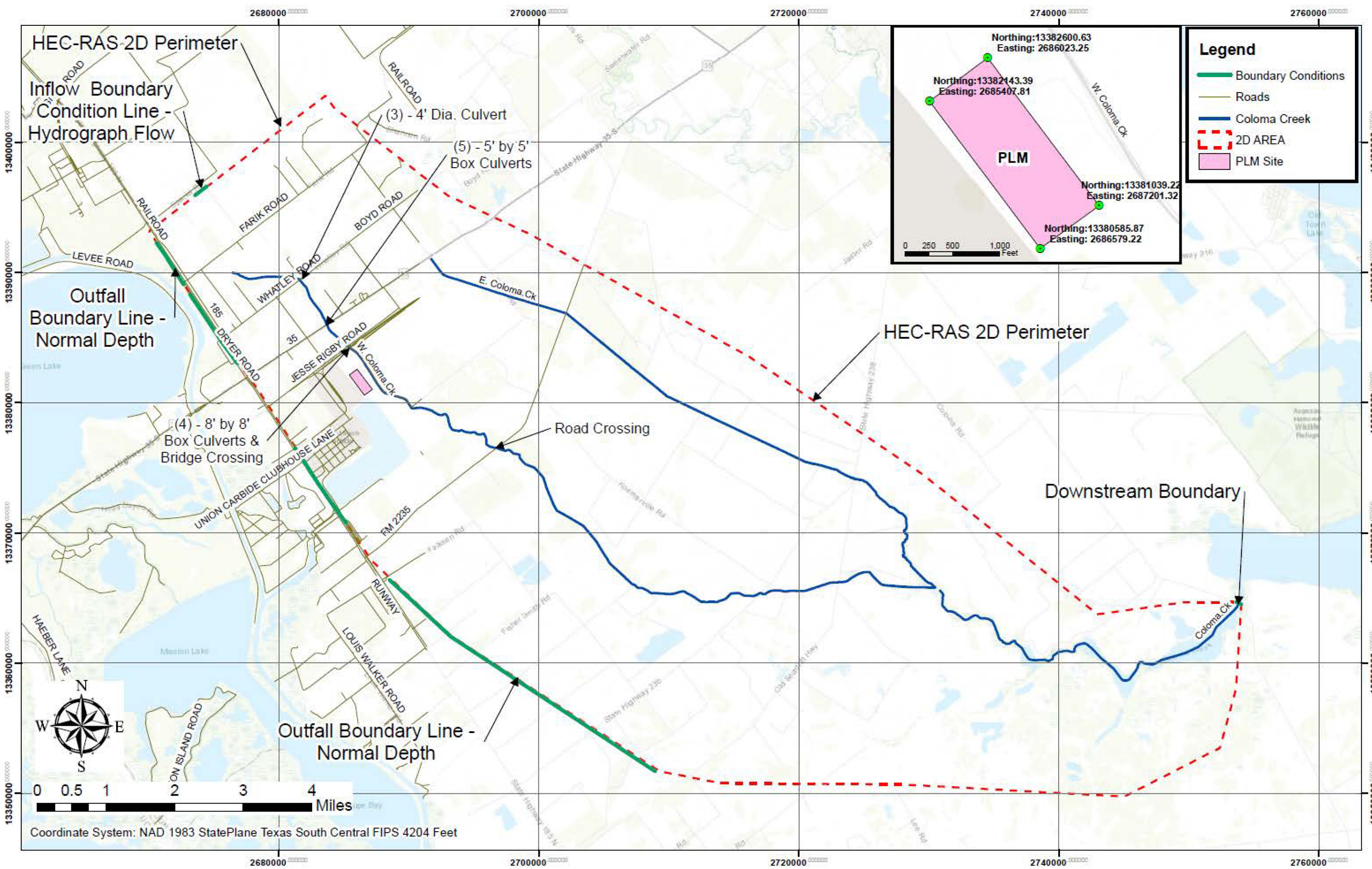


Figure 2.4.3-38
HEC-HMS Model Input/Output for Sparks Watershed

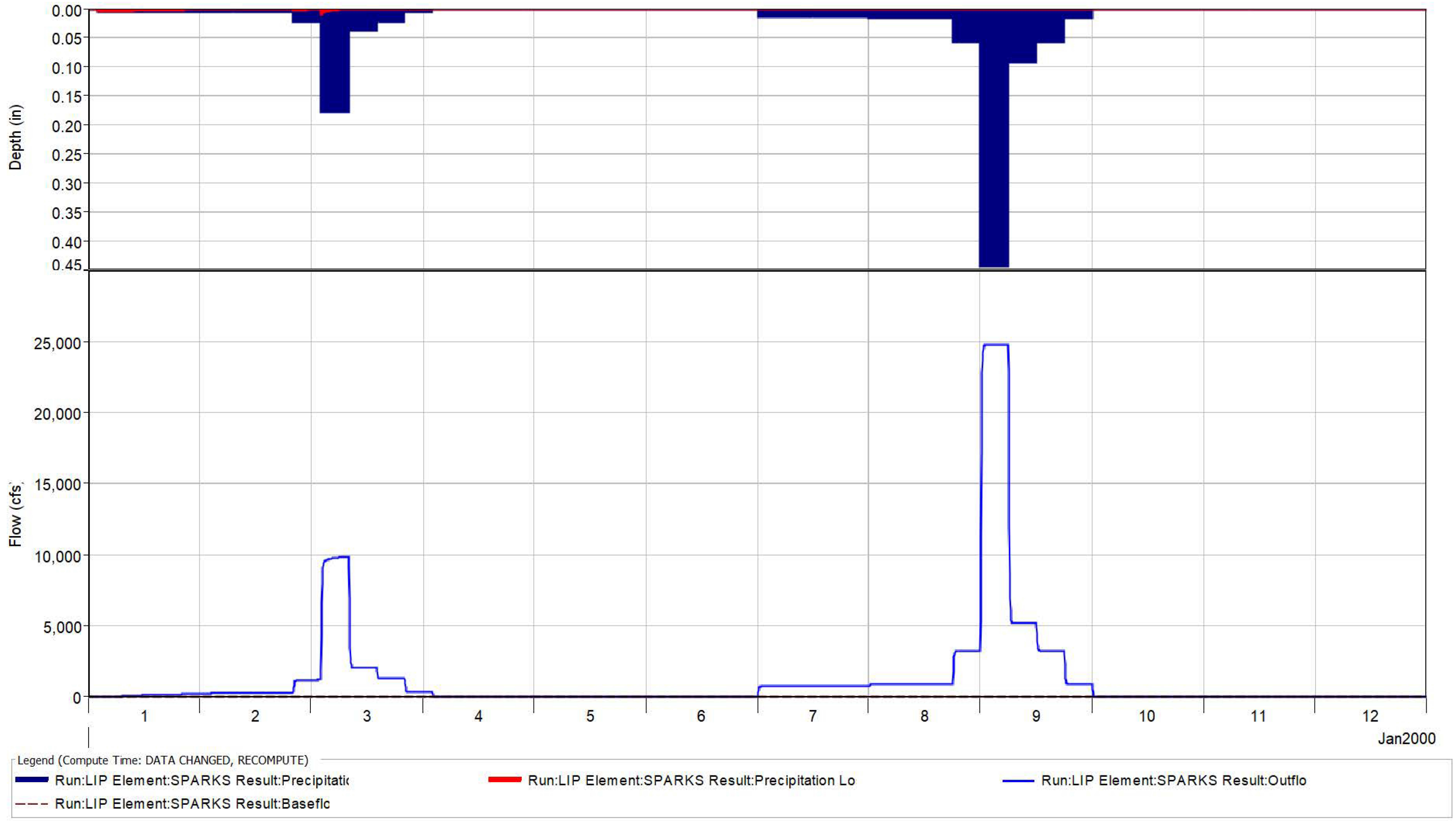


Figure 2.4.3-39
Computational Mesh Overview

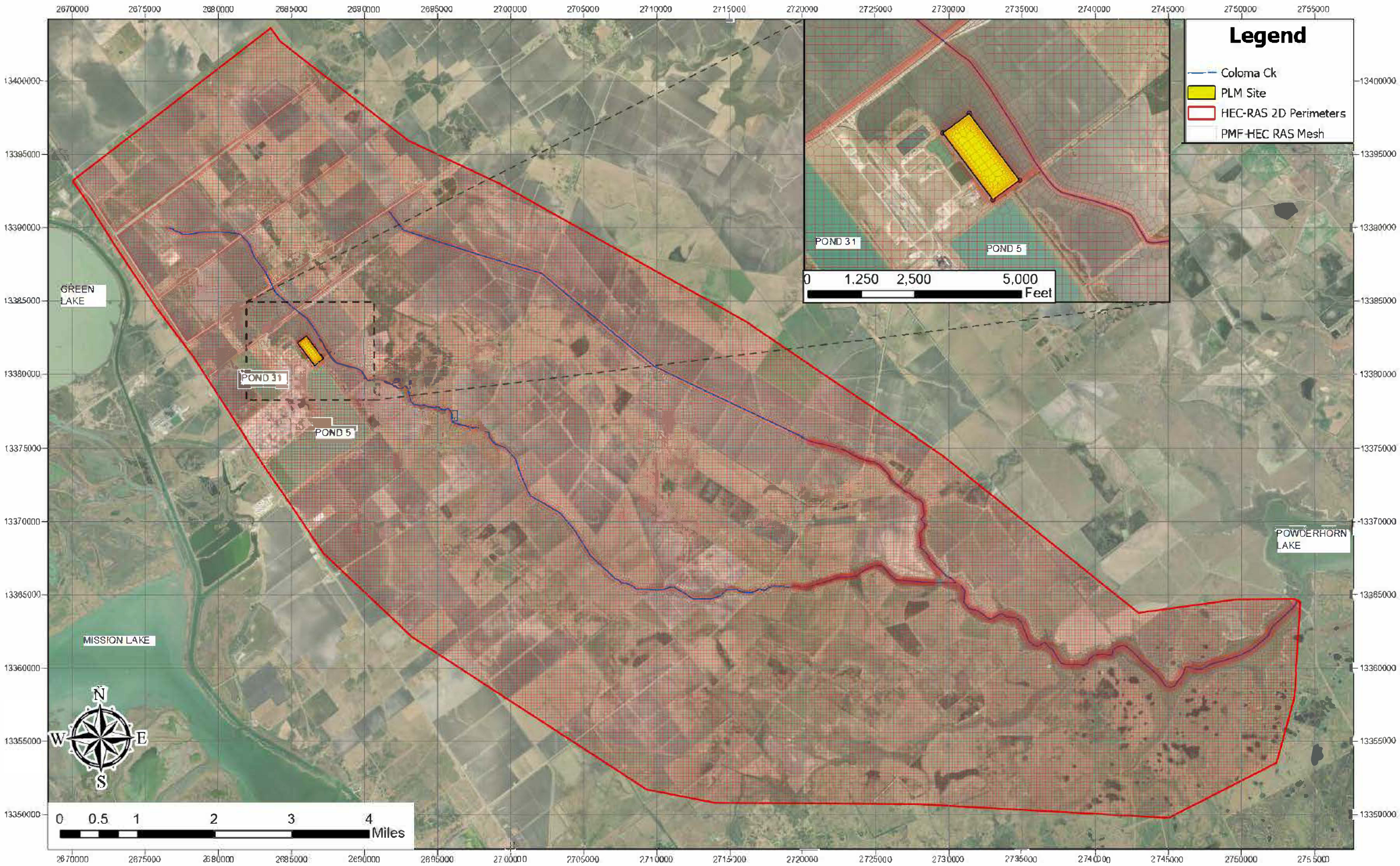


Figure 2.4.3-40
HEC-RAS Unsteady Computation Options and Tolerances

General | 2D Flow Options | 1D/2D Options | Advanced Time Step Control | 1D Mixed Flow Options

☐ Use Coriolis Effects (not used with Diffusion Wave equation)

	Parameter	(Default)	2D AREA
1	Theta (0.5-1.0)	1	1
2	Theta Warmup (0.5-1.0)	1	1
3	Water Surface Tolerance [max=0.2](ft)	0.01	0.01
4	Volume Tolerance (ft)	0.01	0.01
5	Maximum Iterations	20	20
6	Equation Set	Diffusion Wave	Diffusion Wave
7	Initial Conditions Time (hrs)		
8	Initial Conditions Ramp Up Fraction (0-1)	0.1	0.1
9	Number of Time Slices (Integer Value)	1	1
10	Turbulence Model	None	None
11	Longitudinal Mixing Coefficient	0.3	0.3
12	Transverse Mixing Coefficient	0.1	0.1
13	Smagorinsky Coefficient	0.05	0.05
14	Boundary Condition Volume Check	☐	☐
15	Latitude for Coriolis (-90 to 90)		
16	Solver Cores	All Available	All Available
17	Matrix Solver	PARDISO (Direct)	PARDISO (Direct)
18	Convergence Tolerance	0.00001	0.00001
19	Minimum Iterations	3	3
20	Maximum Iterations	30	30
21	Restart Iteration	10	10
22	Relaxation Factor	1.3	1.3
23	SOR Preconditioner Iterations	10	10

Figure 2.4.3-41
HEC-RAS 2D Model Results – Maximum Flow Depth

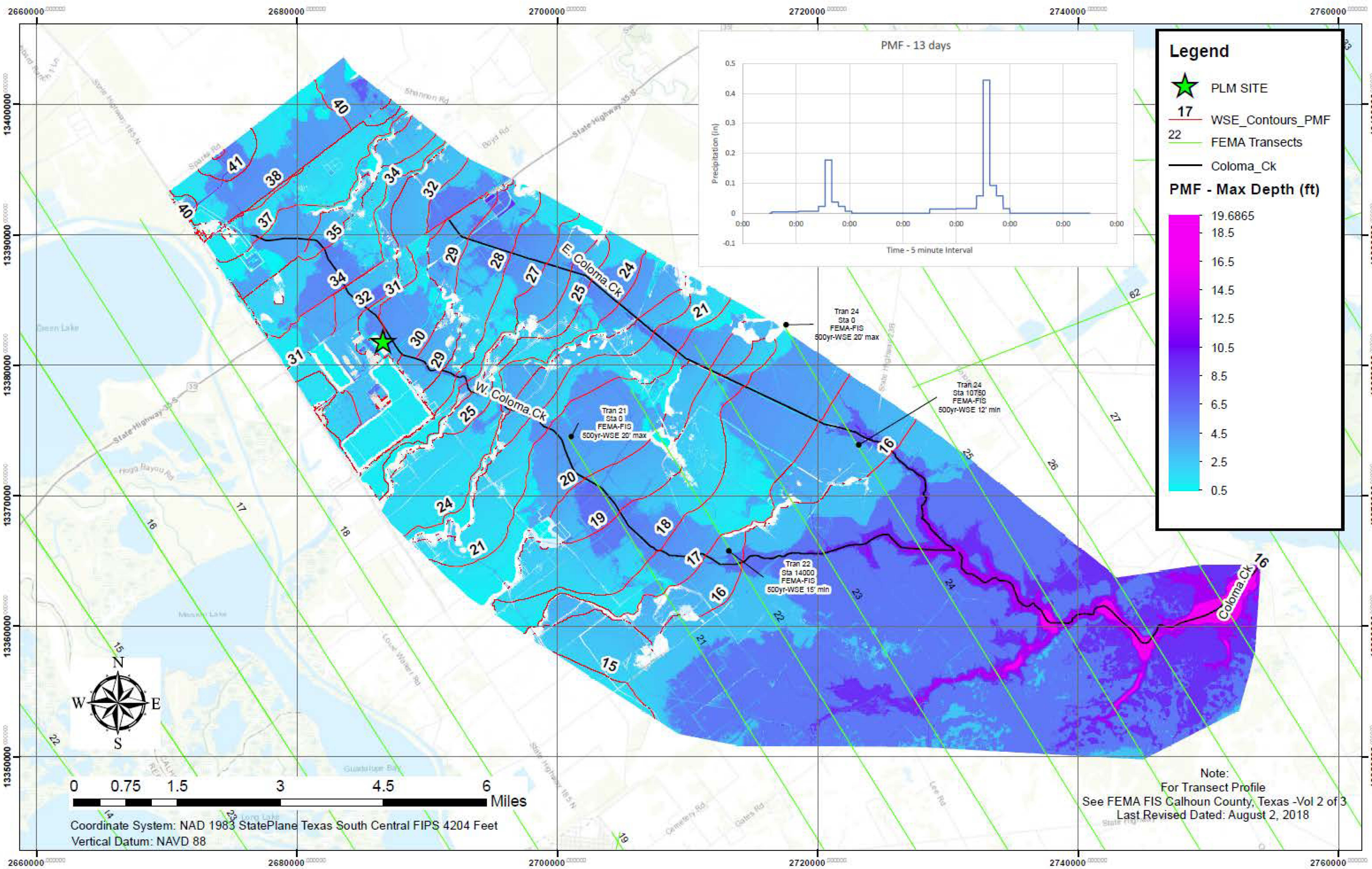


Figure 2.4.3-42
HEC-RAS 2D Model Results – Maximum Flow Depth at Long Mott Generating Station Site

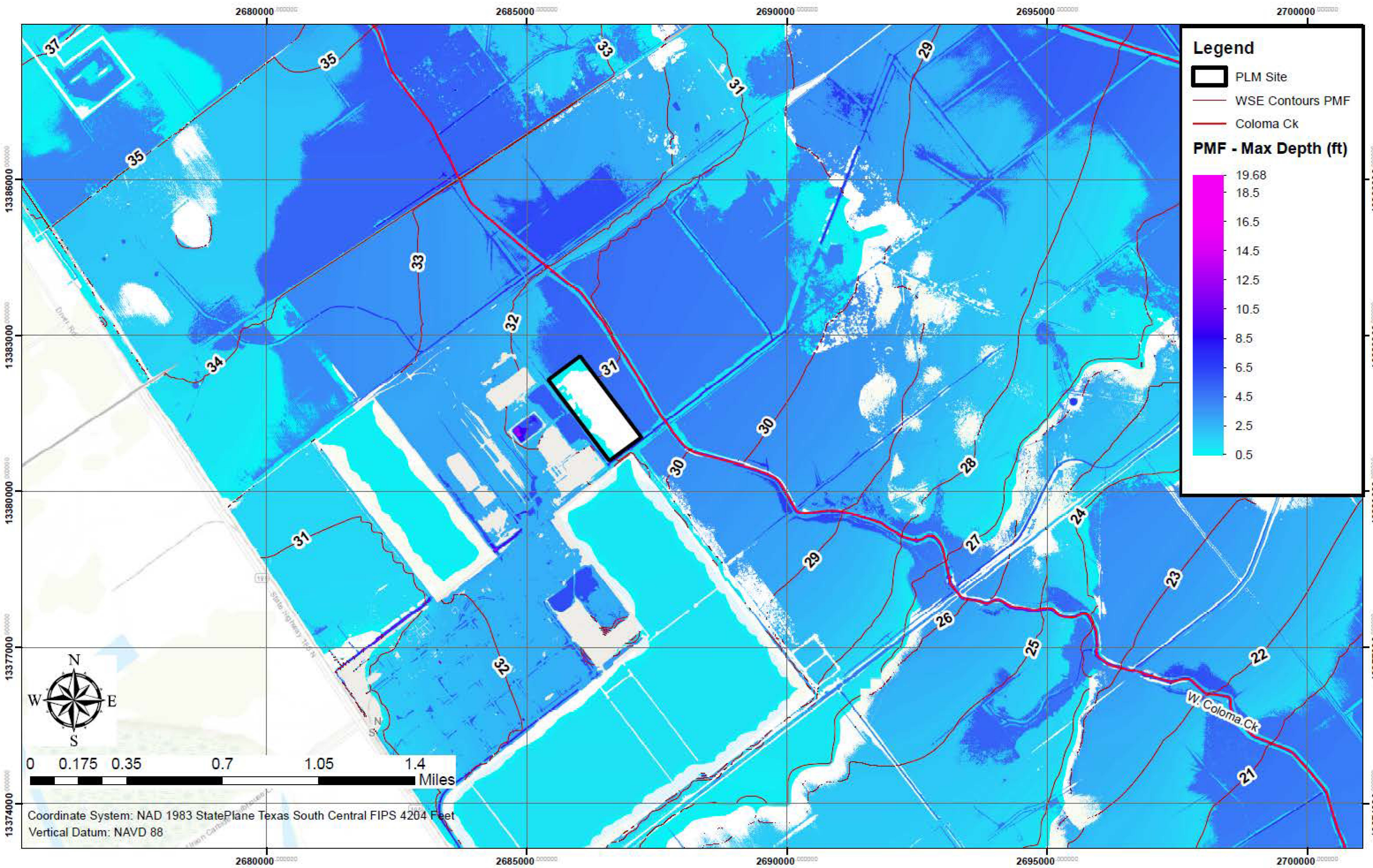


Figure 2.4.3-43
HEC-RAS 2D Model Results – WSEL Upstream of Long Mott Generating Station Site

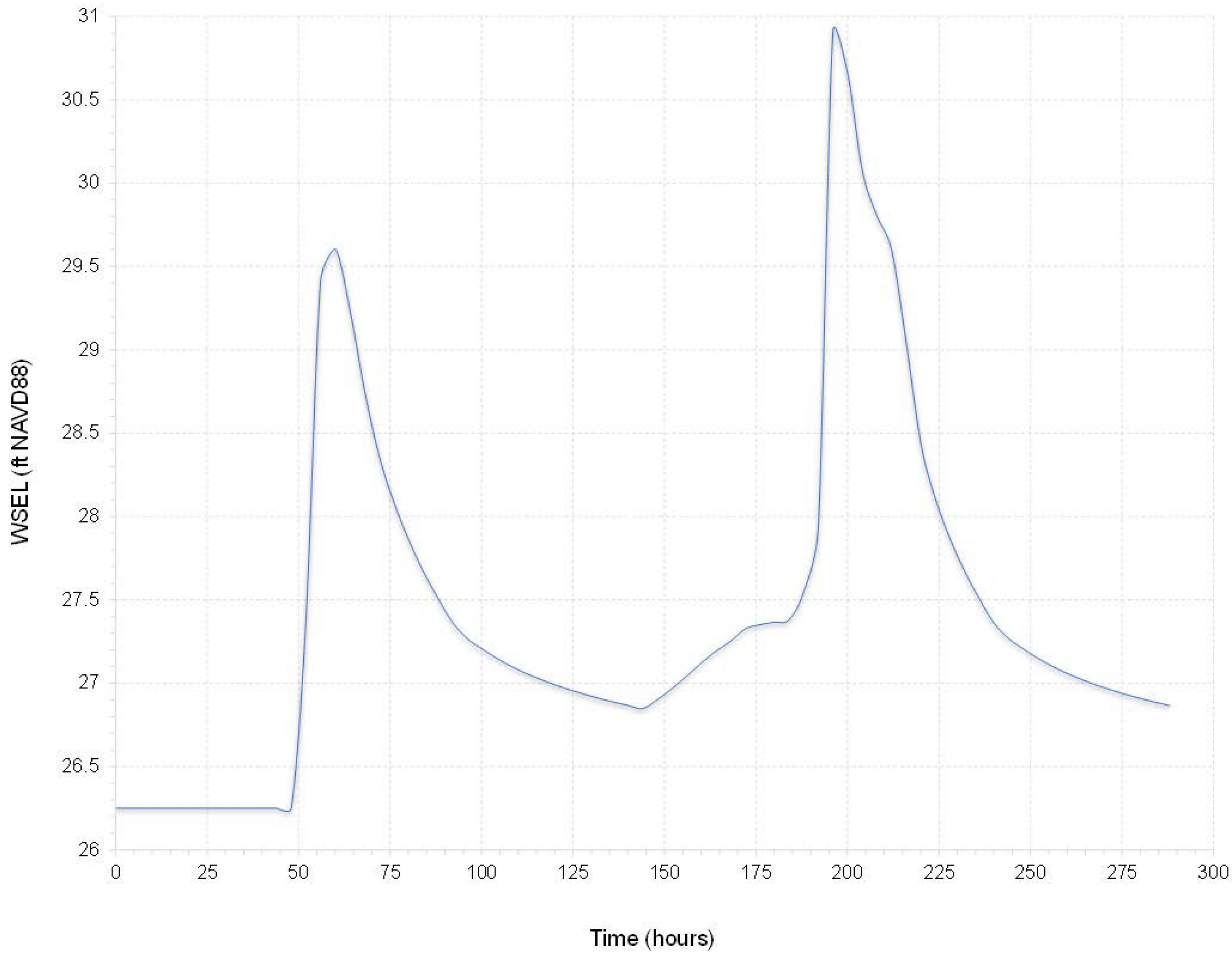


Figure 2.4.3-44
HEC-RAS 2D Model Results – Maximum Flow Velocity

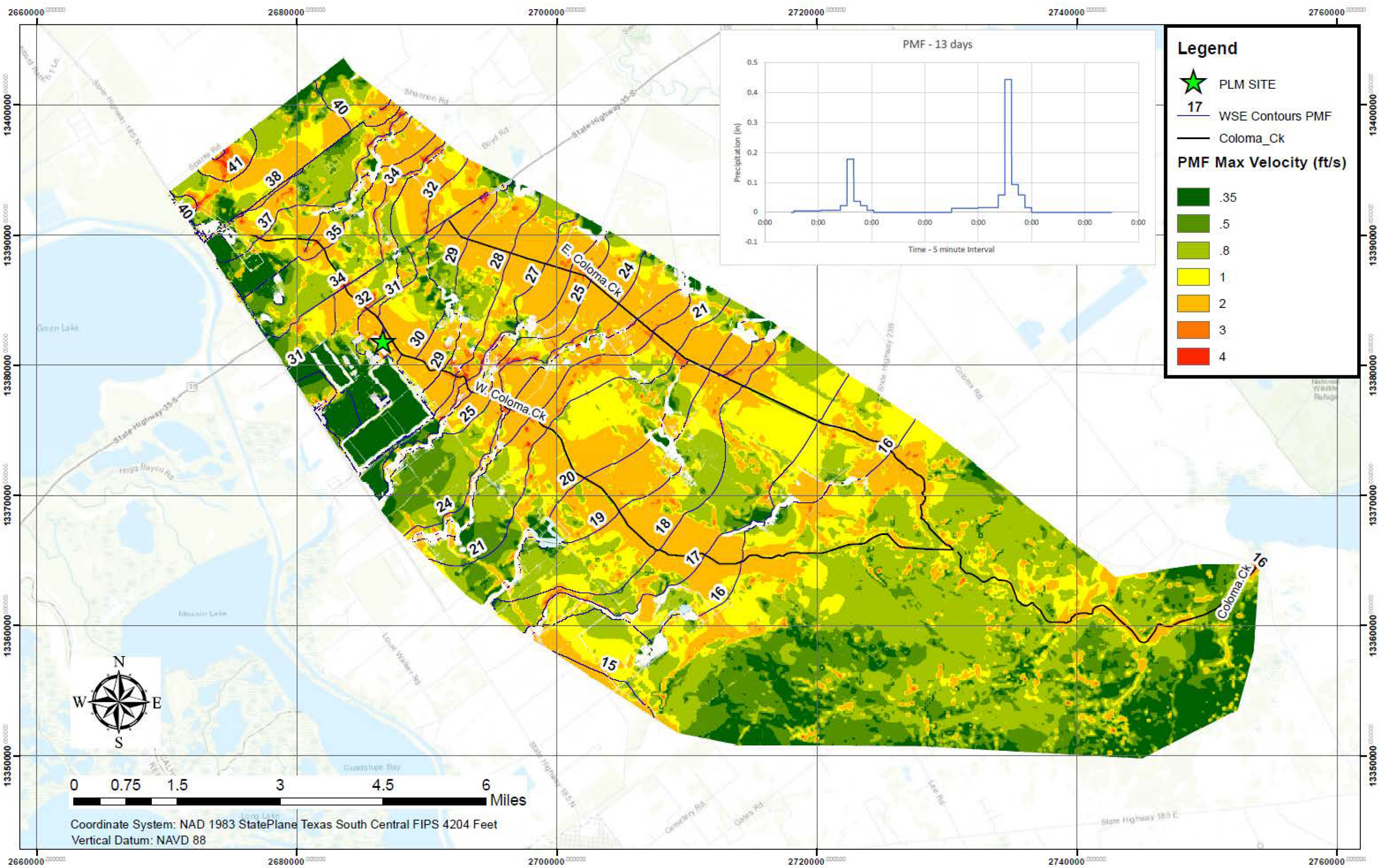


Figure 2.4.3-45
HEC-RAS 2D Model Results – Maximum Flow Velocity at Long Mott Generating Station Site

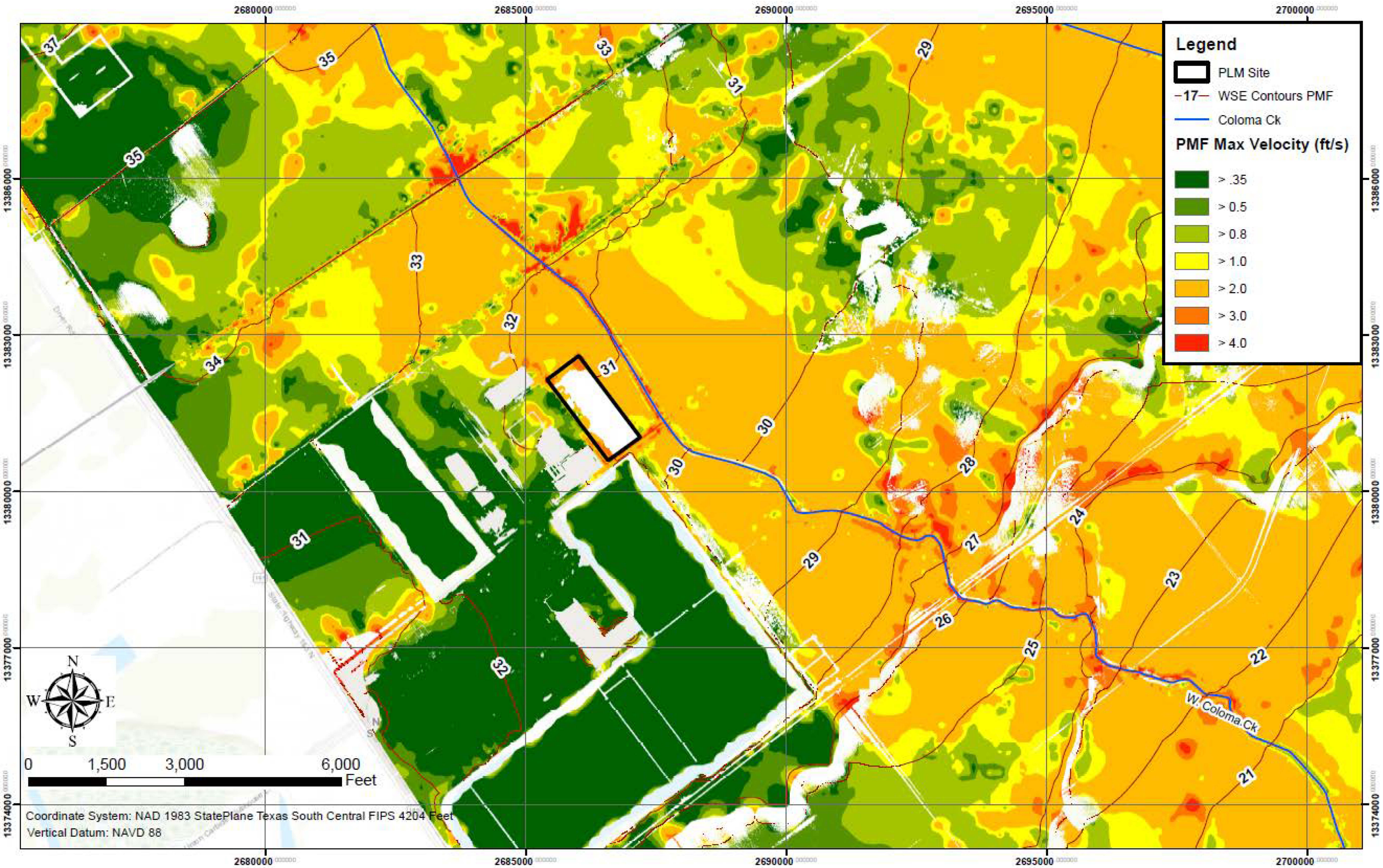


Figure 2.4.3-46
HEC-RAS 2D Model Results – Maximum Fetch

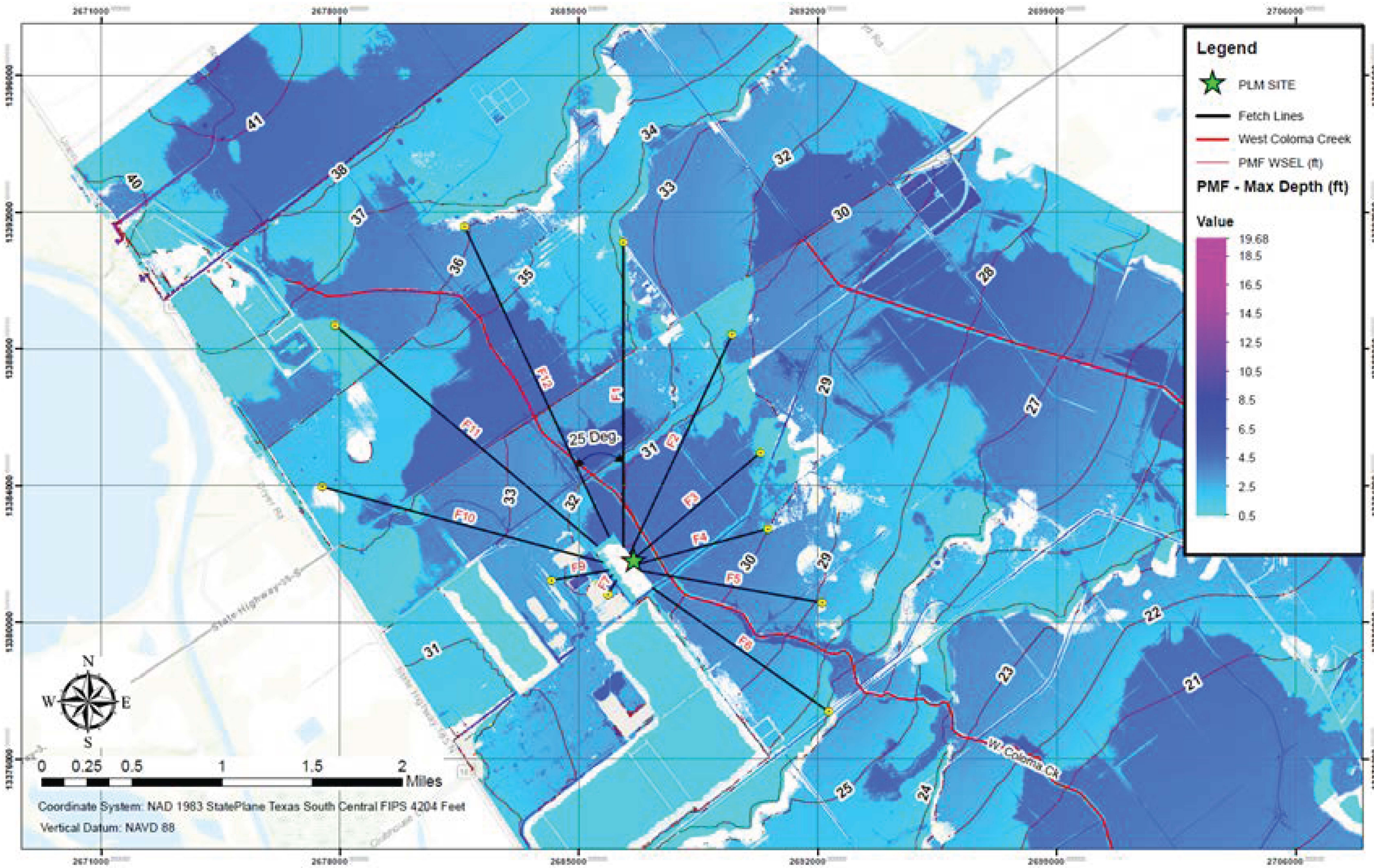


Figure 2.4.3-47
Guadalupe River Basin

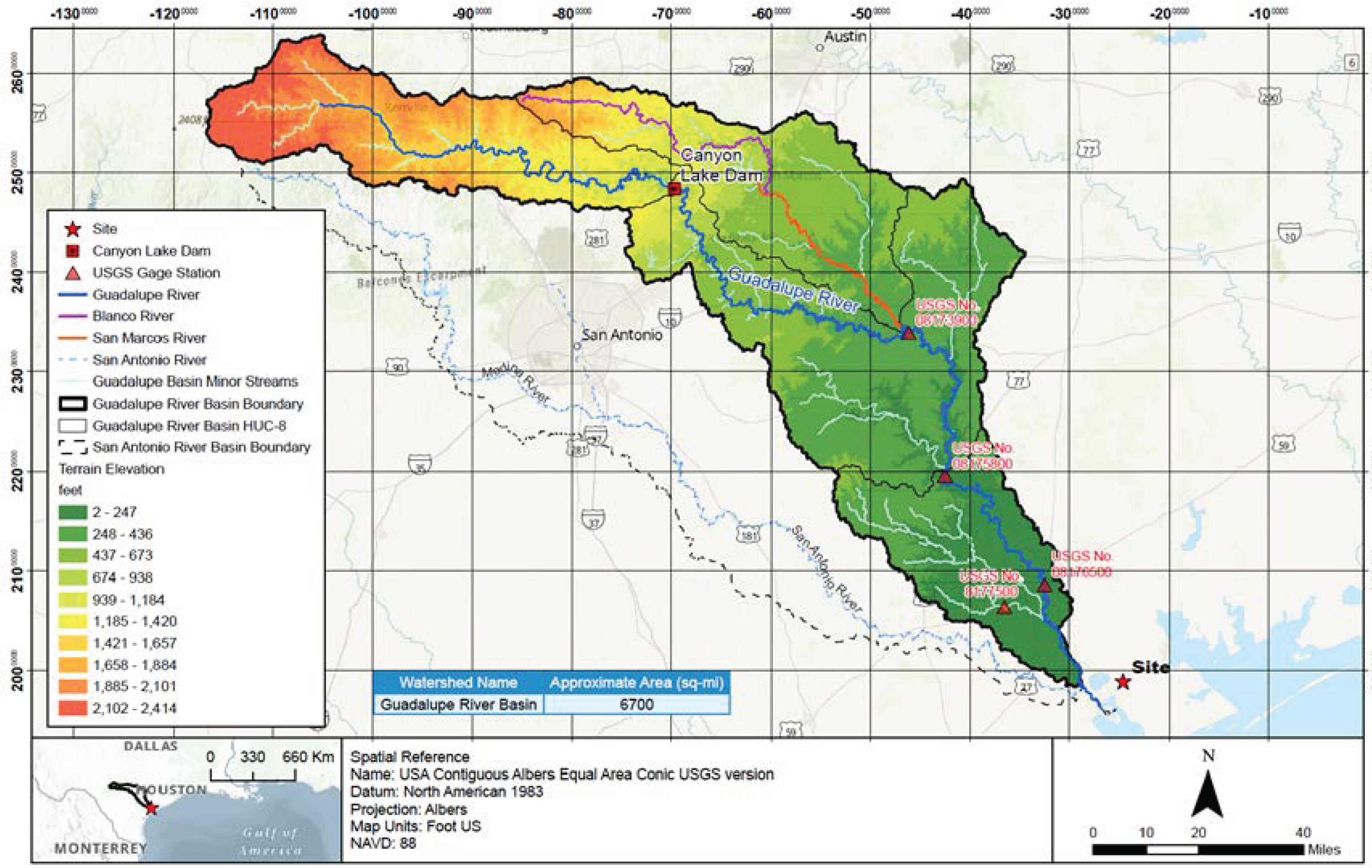


Figure 2.4.3-48
San Antonio River Basin