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Long Mott Energy, LLC

PSAR Subsection 2.4.4, "Potential Dam Failures"

CHAPTER 2

SUBSECTION 2.4.4 POTENTIAL DAM FAILURES

LIST OF TABLES

LIST OF FIGURES

ACRONYMS AND ABBREVIATIONS

Acronym/Abbreviation	Definition
2D	two-dimensional
ac	acre(s)
ACES	Automated Coastal Engineering System
ac-ft	acre-feet
ANSI/ANS	American National Standards Institute/American Nuclear Society
CEDAS	Coastal Engineering Design & Analysis System
cfs	cubic feet per second
cm	centimeter(s)
cms	cubic meters per second
DEM	digital elevation model
FEMA	Federal Emergency Management Agency
ft	feet
ft/s	feet per second
ha	hectare(s)
ha-m	hectare-meter
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Centers River Analysis System

hr	hour(s)
in.	inch(es)
km	kilometer(s)
LMGS	Long Mott Generating Station
m	meter(s)
m/s	meters per second
MHHW	Mean Higher High Water
mi	mile(s)
mi^2	square mile(s)
mph	miles per hour
MSL	mean sea level
NAVD 88	North American Vertical Datum of 1988
NI/CI	Nuclear Island/Conventional Island
NID	National Inventory of Dams
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
PMF	probable maximum flood
SCS	Soil Conservation Service
SLR	sea level rise
SSC	structures(s), system(s) and component(s)

TAC	Texas Administrative Code
TCEQ	Texas Commission on Environmental Quality
TWDB	Texas Water Development Board
U.S.	United States
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USGS	United States Geological Survey
UTM	Universal Transverse Mercator
WSEL	water surface elevation

Chapter 2 Site Characteristics

2.4 HYDROLOGY

2.4.4 POTENTIAL DAM FAILURES

This subsection examines the flooding design basis and evaluates the potential hazards to safety-related structures, systems, and components (SSCs) as a result of potential dam and on-site basin failures.

The LMGS site is located on the east side of the Guadalupe River in Calhoun County, Texas. There are a total of 29 dams on the Guadalupe River and its tributaries and 35~~34~~ dams on the San Antonio River and its tributaries with storage capacity in excess of 3000 ac-ft (370 ha-m) upstream of the LMGS site. These dams and reservoirs are owned and operated by different entities, including the Guadalupe-Blanco River Authority, United States (U.S.) Army Corps of Engineers (USACE), and Natural Resources Conservation Service (NRCS) (previously the U.S. Department of Agriculture (USDA), Soil Conservation Service [SCS]). Figure 2.4.1-4 and Figure 2.4.1-5 show the locations of the dams and storage reservoirs. Specific information on these dams relevant to the flood risk assessment of the LMGS site is summarized in Table 2.4.1-1 and Table 2.4.1-2 based on data collected from the Texas Commission for Environmental Quality (TCEQ).

In Texas, both private and public dams are monitored and regulated by the TCEQ under the Dam Safety Program. Existing dams, as defined in Rule §299.1 of Title 30 of the Texas Administrative Code (TAC) (TAC, 2022), are subject to periodic re-evaluation in consideration of continuing downstream development. Hydrologic criteria contained in Rule §299.15 of Title 30 (Table 3), Hydrologic Criteria for Dams, are the minimum acceptable spillway evaluation criteria for reassessing dam and spillway capacity for existing dams to determine whether upgrading is required. In the area of structural considerations, evaluation of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems (e.g., seepage, cracks, slides, and conduit and control malfunctions) and other structural and maintenance deficiencies that could lead to failure of a structure.

A d~~D~~am failure analysis ~~presented are based on the early site permit application analysis performed for the Victoria County Station (VCS, 2007).~~was conducted for LMGS using guidance found in ~~This reference does not necessarily address the~~ JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards Due to Dam Failure,"~~,~~ published by the U.S. Nuclear Regulatory Commission (NRC) on July 29, 2013 (NRC, 2013).~~as it was issued after the VCS analysis was performed. Additional site-specific analyses and associated information that includes a confirmatory analysis conducted in accordance with the JLD-ISG-2013-01 guideline will be provided by the end of 2025.~~ The analysis concluded that the site does not experience flooding from upstream dam failures with waves considered. The left bank of the Guadalupe River at LMGS is 41.72 ft (12.71 m) North American Vertical Datum of 1988 (NAVD 88). Because the maximum water surface elevation at the site resulting from upstream dam failure is lower than 41.72 ft NAVD 88, the site is protected from flooding in this scenario. ~~. However, it is anticipated that the site will not experience flooding from dam failures given that the combined storage capacity of Canyon Dam and Coleto Creek represents 85 percent of the total storage volume across the 29 reservoirs within the Guadalupe River Watershed and the 35 dams within the San Antonio Watershed are significantly smaller than those in the Guadalupe Watershed~~

~~and collectively account for only 8.5 percent of the total storage capacity of the Guadalupe reservoirs.~~

The postulated cascade failure of the major upstream dams within ~~on~~ the Guadalupe River basin with wave condition accounted for was ~~is~~ considered in the dam failure analysis~~the .~~ ~~VCS, 2007, analysis.~~ Selection of dams for this analysis considered existing upstream dams as documented by TCEQ.

A detailed site-specific breach of the embankments of the on-site operating basins (i.e., basins 5 and 31 in Figure 2.4.4-1) is also analyzed.

These two analyses form the basis of the maximum flood level evaluation for LMGS resulting from potential dam failures. ~~There are no new dams planned in the Guadalupe River basin through 2057, according to the 2007 Texas water plan from the Texas Water Development Board (TWDB) (TWDB, 2007).~~

The Lower Guadalupe diversion dam and saltwater barrier west of the LMGS site is used to restrict saltwater intrusion moving upstream from the Gulf of Mexico during periods of low flow in the river or high storm tides and is not used to supply water for the safety-related SSCs of the plant.

The vertical datum~~Data~~ used ~~in VCS, 2007,a~~ was ~~gathered from different sources that adopted different elevation datums.~~(NAVD 88~~)~~. ~~Because the elevation differences between mean sea level (MSL) (the same as National Geodetic Vertical Datum of 1929 [NGVD 29] and North American Vertical Datum of 1988 [NAVD 88]) are small at Canyon and Coleto Creek Dams, no conversion was made between the datums (NOAA, 2008).~~A datum conversion between NAVD 88 and Mean Sea Level (MSL) and Mean Higher High Water (MHHW) was calculated for wave runup based on the datum analysis results provided by the National Oceanic and Atmospheric Administration (NOAA) at Rockport, Texas (Station Number 8774770) (NOAA, 2024). To convert elevations from MHHW to NAVD 88, 1.3 ft were added to the MHHW elevation. To convert elevations from MHHW to MSL, 0.18 ft were added to the MHHW elevation.

2.4.4.1 Dam Failure Permutations

The reservoirs on the Guadalupe and San Antonio Rivers were reviewed for impact of potential failures based on reservoir size. The Canyon Dam reservoir on the Guadalupe River Basin has a maximum storage of 1,129,300 ac-ft (139,297 ha-m), while the largest reservoir on the San Antonio River, Medina Lake, has a maximum storage of 327,250 ac-ft (40,366 ha-m), roughly one-third the size of Canyon Dam. Reservoirs on the Guadalupe River are notably larger than those on the San Antonio River. Additionally, the combined volume of the 29 dams on the Guadalupe River and its tributaries is 1,482,961 ac-ft (182,921 ha-m), while the combined volume of the 35 dams on the San Antonio River and its tributaries is 647,405 ac-ft (79,856 ha-m), roughly one-half the volume of the Guadalupe River dams.

Therefore, it is concluded that dam failure in the Guadalupe River Basin results in the largest potential ~~probable maximum~~ dam failure at the LMGS site. Simultaneous failure of dams on both basins is not a reasonable scenario and not considered.

2.4.4.1.1 Failures of Upstream Dams in the ~~Gaudalupe~~Guadalupe River Basin

Within the~~Of all the dams on the~~ Guadalupe River Basin, dams upstream of the LMGS site with a capacity over 3000 ac-ft are considered in the dam breach analysis scenarios. Reference JLD-ISG-2013-01, Section 3.1 recommends dam screening on Inconsequential Dam with the following guidance: “Those dams identified by the USACE as meeting the requirements described in Appendix H, ‘Dams Exempt from Portfolio Management Process’ of ER 1110-2-1156, ‘Safety of Dams- Policy and Procedures,’ (USACE, 2011) may be removed from consideration for site impacts.” ~~the~~ Canyon Dam ~~would~~ generates the most significant dam break flood risk on the river. Further, the JLD-ISG-2013-01 states that “Dams identified by federal or state agencies as having minimal or no adverse failure consequences beyond the owner’s property may be removed from further consideration.”

The two largest dams in the watershed are Canyon Dam and Coleto Dam, both of which are considered in the dam failure scenarios. Canyon Dam has the largest dam height of 219 ft (66.8 m) and the largest reservoir storage capacity of about 1.13 million ac-ft (139,383 ha-m) at its peak. The Coleto Creek Dam (on Coleto Creek, which is a tributary of the Guadalupe River), located about 36 river mi (58 km) upstream of Tivoli, Texas, is closest to the LMGS site among all major upstream dams. The Coleto Creek Dam has a height of 65 ft (19.8 m) and a top-of-dam storage capacity of 0.13 million ac-ft (16,035 ha-m).

The following f~~Two separate f~~ailure scenario~~s~~ is ~~are~~ postulated for the upstream dams:

~~Scenario No. 1:~~ Potential dam failures upstream of LMGS were identified in accordance with the JLD-ISG-2013-01 (NRC, 2013). This included determining the combined flooding effects of system and non-system dam failures, which include the peak water surface elevations at LMGS due to potential upstream dam failures. ~~Failure of Canyon Dam induced by a non-hydrologic event.~~ The failure is to occur coincidentally with a 500-year~~.~~ flood or a one-half probable maximum flood (PMF) on the Guadalupe River, in accordance with guidance from the American National Standards Institute (ANSI) and American Nuclear Society (ANS), ANSI/ANS-2.8-1992 (ANSI/ANS, 1992). A separate analysis occurred to determine wind setup, wind-wave run-up, and hydrostatic and hydrodynamic forces on safety-related SSCs at LMGS due to the dam failure flood hazard. The methodology followed analysis procedures described in the USACE Coastal Engineering Manual (USACE, 2013). The maximum wave runup height analysis was added to the maximum water surface elevation from the upstream dam breach analysis for the total breach water surface elevation.

The following analysis steps were taken to assess the failure scenario postulated for the upstream dams:

1. Screen dams in the Guadalupe River basin by removing noncritical dams that have a storage capacity under 3000 ac-ft and low hazard rating from the National Inventory of Dams (NID). Identify the remaining system and non-system dams. Develop hypothetical dams at various locations in the watershed that represent the total volume of the remaining non-system dams at their maximum pool levels, where each hypothetical dam represents a cluster of non-system dams.

2. Determine the dam parameters for the system and non-system dams using the Froehlich 2008 method. Use existing Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model to produce breach outflow hydrographs at the system and

hypothetical dams and route those hydrographs to the mouth of their respective tributaries at the Guadalupe River.

3. Use the existing Hydrologic Engineering Center-River Analysis System (HEC-RAS) unsteady flow model to route the hypothetical dam failure hydrographs from the mouths of the tributaries to LMGS.

~~1.~~4. Compare the predicted total peak water surface elevation at LMGS for the system dam failures, non-system dam failures, and wave runup with the left bank of the river top of levee elevation 41.72 ft (NAVD 88).

~~Scenario No. 2: Failure of Coleto Creek Dam induced by a non-hydrologic event. The failure is to occur coincidentally with a 500-yr. flood or a one-half PMF on Coleto Creek, in accordance with ANSI/ANS-2.8-1992 (ANSI/ANS, 1992)~~

~~Six upstream dams-Lake Meadow, Lake Placid, Lake McQueeney, Lake Dunlap, Lake Gonzales, and Lake Wood, located downstream of Canyon Dam-were not included in the dam break analysis because their dam heights and potential flood volumes would have an insignificant impact on the flood risk compared with Canyon Dam. Their combined maximum storage capacity is about 7.5 percent of that of Canyon Dam, as shown in Table 2.4.1-1.~~

~~The remaining 21 “off-channel” dams are located on the tributaries of the Guadalupe River downstream of Canyon Dam. These off-channel storage dams were also assumed to have no effect on the maximum dam break flood level at the Guadalupe River, as compared with the major dams on the main stem of the Guadalupe River and Coleto Creek.~~

~~Upstream dam failures induced by hydrologic causes such as a PMF is not the controlling scenario as Canyon and Coleto Creek Dams (USACE, 2005a and URS, 2003, respectively) were designed to accommodate and sustain their respective PMFs in accordance with the hydrologic criteria for dams as defined in Rule §299.14 Title 30 of the TAC (TAC, 2022).~~

~~2.4.4.1.6~~2.4.4.1.2 Failure of On-Site Water Control or Storage Structures

Operating basins #5 and #31 are in the vicinity of the LMGS Nuclear Island/Conventional Island (NI/CI) (Figure 2.4.4-1; the basin labeled Basin #5 actually consists of a several adjacent basins treated as a single basin for modelling purposes and collectively referred to as “Basin #5” in this analysis). They are enclosed by a rolled-earthen embankment and rise an average of 10 ft (3.05 m) above the natural ground. The centerline of the north embankment of Basin #5 is adjacent to the south of the LMGS nuclear island. The centerline of the east embankment of Basin #31 is approximately 2850 ft (869.~~,~~7 m). from the western edge of the LMGS site. The natural grade in the vicinity of both embankments varies from 27 ft (8.2 m) NAVD 88 to 30 ft (9.1 m) NAVD 88. Conservatively, a 28 ft (8.5 m) NAVD 88 grade is used along the entire embankment. The top of the embankment is at 38 ft (11.6 m) NAVD 88 with the design pool level of 34.5 ft (10.5 m) NAVD 88. The top of the embankment, which includes 3.5 ft (1.1 m) above normal pool level, is conservatively used in the embankment breach assessment.

Postulated failure mechanisms of the earth embankment include excessive seepage from piping through the foundations of the embankment, seismic activity leading to potential liquefaction of the foundation soils, and erosion of the embankment as the result of overtopping from flood or wind-wave events.

Failure of the operating basins embankments as the result of any of these mechanisms is not considered a credible event. Nevertheless, a conservative approach was adopted in the flood risk evaluation to assume that the embankments would fail. Conservatively, simultaneous failure of both basins is considered. The ~~latest~~ 2008 version of Froehlich's equation (Froehlich, 2008) is used to calculate the peak and formation time. Preliminary calculations showed that the overtopping mode of failure results in a larger peak discharge.

~~2.4.4.1.7~~2.4.4.1.3 Potential for Landslide and Waterborne Missiles

The potential for a major landslide, and hence blockage of the Guadalupe River in the vicinity of the LMGS site, is highly improbable because of wide and flat floodplains and a mild side slope of the river valley.

The potential for waterborne missiles reaching the LMGS site as the result of upstream dam failure is not critical because the site is located above the floodplain of the Guadalupe River, and no flood flow is expected at the site.

There is a potential for waterborne missiles from debris as the result of the embankment breach of the operating basins. However, the flood wave velocity would be attenuated by the time it reaches the safety-related SSCs in the NI, and any debris laden in the flood flow is not considered critical to the design of the safety-related SSCs compared with tornado missiles.

2.4.4.2 Unsteady Flow Analysis of Potential Upstream Dam Failures

2.4.4.2.1 Guadalupe River Basin Dam Failure Modeling~~s~~

Dam Screening

The dams on the Guadalupe River are discussed in Subsection 2.4.4.1. Table 2.4.1-1 lists the height, length, top-of-dam storage capacity, type, and year of completion of the 29 dams with a top-of-dam storage capacity greater than 3000 ac-ft (370 ha-m) each. ~~Of these 29 dams, Canyon and Coleto Creek Dams were selected for inclusion in the dam break analysis.~~ Dams with less than 3000 ac-ft (370 ha-m) storage capacity (i.e., less than 0.25 percent of that of Canyon Dam) were excluded from further evaluation because the impact of their potential breaching on the flood risk at the site would be minimal~~, even if included in the dam failure analysis of the Canyon Dam.~~. Of these 29 dams, all were selected for inclusion in the dam break analysis because they are classified as high hazard dams per the USACE NID. The JLD-ISG-2013-01 states that “Dams identified by federal or state agencies as having minimal or no adverse failure consequences beyond the owner’s property may be removed from further consideration” (NRC, 2013). The dams are divided into system and non-system dams. The system dams are those that fall within the main stem of Guadalupe River and the non-system dams are the dams on its tributaries. Out of 29 dams within watershed, eight (i.e., dams numbered 1, 2, and 7 through 12) are classified as system dams and the rest (i.e., 21 dams) are classified as non-system dams.

The eight system dams considered are the Dunlap, McQueeney, Placid, Meadow, Gonzales, Wood, Canyon, and Coleto dams. Section 3.2 of the JLD-ISG-2013-01 defines a Method 4 Simplified Modeling Approach - Hydrologic Model Method to identify non-critical dams for the remaining non-system dams that have a storage volume above 3000 ac-ft and a high hazard rating from the USACE NID. The non-system dams are clustered by tributary watershed near their confluence with the Guadalupe River or its main tributary. Cluster A is the tributary watershed for Comal River that encompasses all the upstream dams on the tributary. Cluster B is the tributary watershed for Plum Creek, and Cluster C is the tributary watershed for York Creek and the San Marcos River. Each cluster of non-system dams develops a hypothetical dam that represents the total volume of the non-system dams in their tributary watershed at their maximum pool levels. Topographic data in the form of a10-m digital elevation model (DEM) is used to develop an elevation-storage relationship for each hypothetical dam. This elevation-storage curve was then used to determine the starting water surface elevation of the hypothetical dam corresponding to the cumulative storage of all upstream dams represented by the individual hypothetical dam. The invert elevation of the hypothetical dam was derived from the 10-m DEM data and subtracted from the starting water surface elevation to determine the height of the hypothetical dam.

The elevation-area-volume table for a cluster represents the elevation-area-volume of the cluster's tributary watershed. The top elevation of the hypothetical dam is the elevation on the elevation-volume curve for the tributary watershed that corresponds to the total volume of dams in the cluster reported in the NID.

~~Conceptual Unsteady Flow Model~~

~~The dam breach option of the USACE Hydrologic Engineering Centers River Analysis System (HEC-RAS) version 3.1.3 (USACE, 2005b) was used in VCS, 2007, to simulate the dam breach flood waves, which were then routed downstream using the unsteady flow option of the program. Two simulations were performed-one for Canyon Dam and one for Coleto Creek Dam. Details are as follows based on VCS, 2007:~~

~~In the conceptual dam break flood model of Canyon Dam, the dam would fail with the reservoir water level 3 ft (0.9 m) above the dam crest, at elevation 977.0 ft (297.8 m) NAVD 88, thereby releasing the flood storage. In accordance with the combined events requirements specified in ANSI/ANS-2.8-1992 (ANSI/ANS, 1992), the evaluation of potential flood risks as a result of non-hydrologic dam failures should also consider a coincidental flood event equal to a 500-yr. flood or one-half PMF, whichever is less. In this analysis, a constant flood flow of 571,300 cfs (16,177 cms) was conservatively used to represent the coincidental flow. This value is greater than the one-half PMF at a location upstream of the LMGS site (561,650 cfs or 15,904 cms). Details of the PMF values are discussed in Section 2.4.3.~~

~~In the conceptual dam break flood model of Coleto Creek Dam, the dam would fail with the water level 3 ft (0.9 m) above the dam crest, at elevation 123.0 ft (37.5 m) NAVD 88, thereby releasing the flood storage. In accordance with the combined events requirements specified in ANSI/ANS-2.8-1992 (ANSI/ANS, 1992), the evaluation of potential flood risks as a result of non-hydrologic dam failures should also consider a coincidental event equal to a 500-yr. flood or one-half PMF, whichever is less. In the VCS, 2007, analysis, a constant flood flow of 464,000 cfs (13,139 cms) in Coleto Creek and 562,000 cfs (15,914 cms) in the Guadalupe River were conservatively used to represent the coincidental flow. The former corresponds to the PMF outflow from Coleto Creek Dam and the latter to approximately one-half of the PMF value (after rounding) at a location upstream of the LMGS site (discussed in Section 2.4.3).~~

Physical Dam Data and Estimates of Breached Sections

To determine the dam failure hydrograph, several parameters were needed to define the breach formation through the dam. These parameters included the breach bottom width, breach bottom elevations, side slopes, breach development time, and the selected failure method.

The empirical method used to predict the basic geometric and temporal parameters of the dam breach was the Froehlich 2008 method (Froehlich, 2008) described in the dam failure JLD-ISG-2013-01 Section 7.2. The Froehlich 2008 method provides guidance regarding breach parameters based on data collected from 74 embankment dam failures. Froehlich proposed a mathematical expression for final breach peak flow, breach width, side slope of a trapezoidal breach, and formation time. The findings of the statistical analysis were applied in a Monte Carlo simulation to estimate the degree of uncertainty (Froehlich, 2008). This study uses the 2008 version of Froehlich's equation to calculate breach width and formation time. A linear progression method for the growth of the breach was selected. In this method, incremental rates for the growth of depth and width remain constant during the entire breach development. A 5-minute time step was used to perform model computations.

The Froehlich, 2008 method suggests using breach side slope factors of 1.4 for overtopping failures and 0.9 for other failure modes. For all system and hypothetical dams, except Canyon Dam, the HEC-HMS overtopping dam failure method is selected, which represents failures caused by overtopping of the dam. For all system and hypothetical dam breach parameters, except Canyon Dam, the failure mode factor used is 1.3 and breach side-slope ratio of 1 for overtopping failures. For Canyon Dam, the failure method is failure mode by piping with a factor of 1 and side-slope ratio of 1. For the hypothetical dams, the maximum volume of water is the total water volume in the cluster.

~~Canyon Dam is an earth dam, located about 270 river mi. (435 km) upstream of the LMGS site, is 6830 ft (2082 m) long and 219 ft (66.8 m) high. Following the guidelines from the Federal Energy Regulatory Commission (FERC) on dam break analysis (FERC, 1993), five times the dam height (1120 ft or 341.4 m) is assumed to be the average breach width in the simulation. The breach section in the model was represented by a 1:1 side slope with bottom and top widths of 896 ft (273 m) and 1344 ft (410 m), respectively. The time to complete the breach was assumed to be 0.1 hr, based on guidelines from the FERC for the estimation of the dam breach parameter (FERC, 1993). The model cross section for Canyon Dam is shown in Figure 2.4.4-2.~~

~~Coleto Creek Dam, an earth dam 18,950 ft (5776 m) long and 65 ft (19.8 m) high, is located about 46 river mi. (74 km) upstream of the LMGS site. The concrete spillway structure has a gross width of 328 ft (100.0 m) and a total width of 408 ft (124 m), including the extensions on both sides into the earthen embankment. Following the FERC guidelines (FERC, 1993), the entire spillway concrete structure is postulated to fail. This is greater than five times the dam height that would fail along the earthen embankment (325 ft or 99 m). The time to complete the breach was also assumed to be 0.1 hr. The model cross section for Coleto Creek Dam is shown in Figure 2.4.4-3.~~

~~Table 2.4.4-1 lists the dam breach characteristics used to model the failure of these two dams.~~

HEC-HMS Model Setup & Modifications

The USACE provided a HEC-HMS model for the Guadalupe River Basin. The meteorological input method used in the original HEC-HMS model was a frequency-based elliptical design storm applied across the entire Guadalupe River Basin. The elliptical storm frequency simulations in HEC-HMS were performed for 60 junctions. The model was then simplified to include only one setup configuration for ease of use. The elliptical storms used the same point rainfall tables and durations as the uniform rainfall method from the NOAA Atlas 14, Volume 11. The point precipitation values that were applied to each elliptical storm were based on the storm center's locations, not the location of the gage outlet of interest. The center point location and the rotation of the major axis for seven gages was used to determine the highest peak for the 500-year storm event. The elliptical storms used a maximum storm extent of 10,000 mi^2, and an elliptical ratio of 3:1. A sensitivity analysis was conducted to determine the gage that produced the largest discharge for the 500-year storm event and covered majority of the lower Guadalupe River Basin. It was determined that rainfall at the Cuero gage produced the highest peak discharge of 773,141 cfs (21893 cms) when compared at the outfall of the model near Bloomington, Texas, shown in Figure 2.4.4-2.

The peak discharge of each hydrograph was adjusted by shifting the timing to correspond with the peak discharge from the selected 500-year elliptical storm event associated with each dam failure location throughout the Guadalupe River Basin.

Parameters used were NRCS deficient and constant losses, recession baseflow, Snyder Unit Hydrograph for transformation, and Lag, Modified Plus, and Muskingum methods for routing. The simulation was run for a total time of 9 days with a computational interval of 15 minutes.

The upstream limit for the HEC-HMS model was set at the outfall of Canyon Dam. Consistent with the configuration of the original hydrology and hydraulic models provided by the USACE, Canyon Dam outflow was disconnected from downstream reaches in all simulations. The outfall element for the HEC-HMS model, labeled "GuadalupeRv nr Bloomington TX," is located approximately 15.5 mi northwest of the site. This element was used as the endpoint outfall for HEC-HMS simulations.

The dam breach hydrographs were added into the existing HEC-HMS model where the peak discharge of each hydrograph was shifted to correspond with the timing of the peak discharge from the selected 500-year elliptical storm event at Cuero throughout the Guadalupe River Basin. The peak discharge of each hydrograph was adjusted by shifting the timing to correspond with the peak discharge from the selected 500-year elliptical storm event associated with each dam failure location throughout the Guadalupe River Basin. The original HEC-HMS modeled reach elements were configured with a maximum discharge capacity of 1,000,000 cfs (28,317 cms). The combination of dam failure and elliptical storm events produced peak flows that exceeded the limits established by the reach rating curves defined in the original calibrated parameters. The original HEC-HMS routing parameters were adjusted, increasing the discharge capacity by 20 percent to 30 percent for most reaches using the Modified Puls method. Notably, one reach element, which simulates the Guadalupe River routing from the Canyon Lake outfall to New Braunfels, had its discharge capacity increased by 60 percent.

HEC-HMS (USACE, 2023a) was used for peak discharge output results to be inputted into the HEC-RAS analysis (USACE, 2023b).

HEC-RAS Model Setup & Modifications~~Channel Geometry~~

For the Guadalupe River, cross section data were obtained from USACE in an existing HEC-RAS model for the Guadalupe River basin. The primary sources of topographic data used in the

HEC-RAS model were developed from the 2007 – 2008 CAPCOG and Texas Natural Resources Information System LiDAR data. 3 ft by 3 ft digital DEMs were generated from the LiDAR data for use in the hydraulic modeling. For river basin areas that extended past the cross sections, the U.S. Geological Survey (USGS) 10-m DEM was used. The HEC-GeoRAS extension is used to cut cross sections from the 3-ft DEMs and to geo-reference existing models. Some channel sections were modified to match field measurements, as built drawings and survey data. Manning's roughness values were developed based on land use maps, aerial photography, and site visits. Field surveys of open channel sections and bridges/culverts along the detailed study reaches of the Guadalupe and San Marcos Rivers were conducted April 2013 through July 2013. Some channel section surveys were collected using boat-mounted sonar equipment where the water was too deep for standard survey methods. Available bridges/culverts for all streams were modeled using field measurements, "as-built" plans, or bridge/culvert data from the current effective USACE models. Where available, survey data was incorporated in the final hydraulic models as well. The omission of the river bathymetric data yields conservative flood level estimates because of the artificially raised bed level and the corresponding reduction in flow capacity.

For the purposes of this study, the upstream limit for the HEC-RAS model was set at the outfall of Canyon Dam, defined as Station 295.00. The HEC-RAS reach model utilized in this assessment had its outfall located approximately 500 ft northwest of Mission Lake in Calhoun County, defined as Station 0.00. For analysis purposes, Station 1.255—located roughly 2000 ft northwest of the LMGS study site—was established as the downstream study limit shown in Figure 2.4.4-3. The downstream boundary condition as the study limit is defined at the normal depth set to a frictional slope of 0.0001. The total model reach length is 295 mi, and the number of cross sections is 1,211.

Modifications were made to cross section 1.255 to capture the LMGS site. To analyze the study site in relation to the observed results, HEC-RAS cross section 1.255, originally measuring 29,470 ft in length, was extended approximately 12,845 ft to the northwest from the endpoint of the original model. This extension concludes just after West Coloma Creek intersects with Jesse Rigby Road and runs parallel to Jesse Rigby Road along its modified length. As a result, the total length of the modified cross section was set to 42,315 ft.

Eleven dam breach hydrographs were incorporated into the dam failure HEC-RAS model to represent various failure points along the Guadalupe River Basin. The dam failure points used in HEC-RAS are shown in Figure 2.4.4-2. There hydrographs were strategically placed at key upstream locations throughout the basin. The eleven hydrographs were arranged into two simulation cases with the 500-year frequency elliptical storm event. The dam breach hydrograph inflow locations were at station 252.376, 172.727, 169.347, or 32.069 for both simulations. For simulation 1, system dam breach hydrographs were added at the following respective stations: station 252.376 for Canyon, Dunlap, McQueeney, Placid, and Meadow dams; station 172.727 for Gonzales and Wood Dams; and station 32.069 for Coleto Dam. For simulation 2, non-system dam breach hydrographs were added at the following respective stations: station 252.376 for cluster A hypothetical dam; and station 169.347 for clusters B and C hypothetical dams. The dam failure hydrographs inputted for cases 1 and 2 are shown in Figures 2.4.4-4 and 2.4.4-5, respectively.

Inflow hydrographs at all other HEC-RAS stations are consistent with the original HEC-HMS model for the 500-year frequency storm event. There is a total of 26 inflow stations, and of the 26 stations, 22 stations remain unchanged. The stations of inflow hydrographs into HEC-RAS are listed in Table 2.4.4-1. Also consistent with the configuration of the original hydrology and hydraulic models provided by the USACE, Canyon Dam outflow was disconnected from downstream reaches in all simulations. The 500-year inflow from the San Antonio River is considered at station 6.04, consistent with the original model. The inflow hydrographs modeled in HEC-HMS were added as boundary conditions in the HEC-RAS model. HEC-RAS was used to determine the water surface elevation at the site in this study..

~~two sources: (1) HEC-RAS models developed for the Federal Emergency Management Agency's (FEMA's) Flood Insurance Study (FIS) along the river (CH2M Hill, 2002, Halff, 2004, and Halff, 2006) and (2)~~

~~U.S. Geological Survey (USGS) National Elevation Dataset (NED) (USGS, 2007). The cross-section data from FEMA's FIS is composed of three reaches:~~

1. ~~Comal County, just downstream of Canyon Dam to the River Road Second Crossing in New Braunfels, Texas, covering 14.5 river mi. (23.3 km) (Halff, 2004).~~
2. ~~Guadalupe County, from the downstream limit of (a) to Dunlap Dam (5 mi. or 8 km below the city limit of New Braunfels, Texas), covering 17.1 mi. (27.5 km) (CH2M Hill, 2002).~~
3. ~~From just upstream of Dunlap Dam to its confluence with Geronimo Creek, approximately 2500 ft (762.0 m) downstream of FM-466, covering 22.8 river mi. (36.7 km) (Halff, 2006).~~

~~All the bridges, redundant cross sections (some associated with the bridges), diversions, and dams in the models were removed. Under the flooding condition of a dam breach, the bridges and dams would most likely be overtopped or washed out; therefore, removing them is a realistic approximation. In addition, removing these structures upstream of the site makes the analysis conservative by eliminating any flow resistance and flood attenuation due to storage.~~

~~The cross sections for the FIS (developed from USGS quadrangle contours, LiDAR data, local topographic maps, and field surveys) were developed to simulate the 100- and 500-yr. floods and are not wide enough to accommodate the more severe flooding expected from a dam breach event. However, HEC-RAS assumes vertical walls at the ends of cross sections if the flood level is higher than the maximum ground level specified for a cross section. This assumption is conservative in that it does not allow for a flood to spread, and, therefore, overestimates the corresponding water level. Thus, the results would represent higher and conservative estimates of the flood level from a dam breach.~~

~~The cross sections for the Guadalupe River downstream of the Geronimo Creek confluence to the downstream end of the model (about 2.5 mi. or 4.0 km upstream of the river mouth) were developed by employing the HEC-GeoRAS version 4.1.1 module in conjunction with ArcView/ArcGIS version 9. The length of this reach is about 239.3 river mi (385.1 km). The basis for the cross sections is the NED model developed by the USGS (USGS, 2007). The NED data (in ESRI raster grid format with 30-m or 98.4-ft resolution) was converted into a triangular irregular network using ArcView/ArcGIS and specified as an input for HEC-GeoRAS processing.~~

~~The cross sections obtained from the USGS NED do not include definitive bathymetry of the river (under water/riverbed topography). However, floods from a dam breach event would be of such magnitude that the channel flow would not be significant compared with total flood flows.~~

~~The omission of the river bathymetric data yields conservative flood level estimates because of the artificially raised bed level and the corresponding reduction in flow capacity. The number of original cross sections obtained from FEMA's FIS and USGS NED were 427 and 103, respectively. The total number of cross sections, including interpolated cross sections, is 1232. Of these, 539 correspond to FEMA's FIS data; the remaining 693 are from USGS NED.~~

~~The locations and extents of the cross sections~~ and inflow ~~used in the HEC-RAS dam break model for the Guadalupe River are shown in~~ SNL, 2025a, Appendix 1. ~~Figure 2.4.4-4.~~

~~For Coleto Creek, cross section data were obtained from two sources: (1) HEC-2 models developed for FEMA's FIS along the creek (FEMA, 1990), and (2) NED data (USGS, 2007). The cross sections from FEMA's FIS cover the area from just downstream of Coleto Creek Dam to the confluence with the Guadalupe River, about 13 river mi (20.9 km), and were developed by Half Associates, Inc., in March 1985. The reach between just downstream of the dam and FM-446 was updated in October 1995 by the U.S. Bureau of Reclamation to incorporate changes in the channel cross sections and bridge details (FEMA, 1998). For this study, the geometric data from the two HEC-2 models was consolidated to develop the Coleto Creek HEC-RAS model. This geometric data was again combined with the cross sections obtained for the Guadalupe River downstream of the confluence with Coleto Creek. Thirty-three of the original HEC-2 model cross sections were adopted from FEMA's FIS, and 23 were adopted from the USGS NED. The total number of cross sections, including interpolated cross sections and excluding the dam, is 212. Out of these, 120 correspond to USGS NED data and 92 to FEMA's FIS data.~~

Wave Runup Analysis

A wave runup analysis was conducted to estimate the contribution of waves to potential upstream dam failure flood levels for LMGS. Design input used is the 2-year mean recurrence interval annual extreme-mile wind speed obtained from ANSI/ANS-2.8-1992, which is 50 mph at LMGS at 30 ft above ground (ANSI/ANS, 1992). The average water depths along various fetch angles are extracted from inundation results of dam failure analysis in HEC-RAS at station 1.255. Areas with a water depth shallower than 1 ft were excluded because wave growth would be insignificant due to depth-limiting conditions.

Wind-driven waves were calculated using the “Windspeed 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). The needed inputs for the calculation of wind-driven waves in 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. 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).

Guidance in CETN I-45 states that the ratio of H_{mo} to H_s can be assumed to be 1 for wave steepness greater than 0.01 (USACE, 1991). The calculation showed that the wave steepness is greater than 0.01; therefore, H_{mo} was assumed to be equal to H_s. In accordance with USACE Engineering Manual EM 1110-2-1614, Equation 2-2 (USACE, 1995) and ANSI/ANS-2.8-1992 (ANSI/ANS, 1992), the design wave height, also referred to 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 approximation that $H_{1\%}$ is 1.67 times H_s.

To calculate wind setup, U.S. Bureau of Reclamation ACER Technical Memorandum No. 2, Equation 4, is used. This document has the following parameters: design wind velocity, wind fetch in miles, average water depth for the fetch angles of true North, East, South, and West are 0, 90, 180, and 270 degrees, respectively (USBR, 1981).

Wave height (H_{mo}), wave period (T_p), wave length, average wave height of the highest 1 percent of waves ($H_{1\%}$), wind setup, and wave runup were all calculated to determine the total dam failure water surface elevation (WSEL) at LMGS. A sea level rise (SLR) of 1.18 ft and stillwater dam failure WSEL of 27.89 ft (NAVD 88) were used for the 4 fetch angles.

~~Bed Roughness Coefficient Used in the HEC-RAS Models~~

~~The HEC-RAS computer program requires roughness coefficients, or Manning's n values, to describe land surface roughness that the river flows experience in the river channel and floodplain. Typically, different roughness coefficients are used to represent the channel and floodplain over-bank areas of each cross section based on land cover and bed material type.~~

~~For the Comal County reach of the Guadalupe River, Manning's n values were assigned by visual inspection and analysis of aerial photographs. The river segment in this study is primarily in non-urban floodplain conditions (unimproved channels and agricultural over-banks with scattered low-density residential areas). The Manning's roughness coefficient for the Guadalupe River varied from 0.035 to 0.040. Over-bank pastures or field areas were assigned Manning's n values between 0.045 and 0.050. Values varying between 0.06 and 0.08 were used for the channel over-banks through developed and treed areas. More densely treed areas were assigned roughness coefficient values as high as 0.10 to 0.14. The model for this reach was calibrated based on historical floods, and details are given in Halff, 2004.~~

~~For the Guadalupe County reach of the Guadalupe River, upstream of Dunlap Dam, the roughness coefficients were visually matched with conditions on the ground based on aerial photographs. For the over-bank flows, Manning's n values ranged from 0.04 for pastures with no brush and agricultural land with row crops to 0.08 for houses with trees. For channel flows, the Manning's n values ranged from 0.035 to 0.040. The model for this reach was calibrated based on historical floods and details are given in CH2M Hill, 2002.~~

~~For the Guadalupe County reach of the Guadalupe River, downstream of Dunlap Dam, Manning's n values were assigned by visual inspection and analysis of aerial and field photographs (Halff, 2006). Channel n values range from 0.02 to 0.10, and over-bank n values range from 0.035 to 0.100. In general, Manning's roughness coefficient for grassy, weedy channels to weedy with trees and pools ranged from 0.04 to 0.06. Over-bank pastures or field areas were assigned n values between 0.035 and 0.055. Values varying between 0.06 and 0.08 were used for the channel over-banks for developed and treed areas. More densely treed areas were assigned roughness coefficient values as high as 0.10. The model for this reach was calibrated based on historical floods and details are provided in Halff, 2006.~~

~~For sections that were developed using the USGS NED data, Manning's n value of 0.1 was adopted conservatively to the entire extent in each of the cross sections. This corresponds to the upper range of over-bank roughness values used in the calibrated flood studies summarized above.~~

Predicted Water Levels from Upstream Dam Failure

The peak discharge hydrographs from the 500-year elliptical storm event at Cuero and dam breach hydrographs were routed in HEC-HMS and inputted into HEC-RAS for the dam failure analysis.

For the HEC-RAS dam breach simulations the downstream boundary condition is assumed to be at normal depth with a slope equal to 0.0001~~6~~.~~, as discussed in Section 2.4.3.~~

The peak water surface levels are recorded at station 1.255 for the site. The time elapsed to reach the estimated peak stage is 2 days, 11 hours, and 15 minutes. The unsteady flow routing for the 500-year flood event at Cuero alone in HEC-RAS results in water surface elevation of 24 ft~~.~~ NAVD 88 from the Guadalupe River at the LMGS site. For the non-system dam failure, the water surface elevation is 24.72 ft (7.53 m) NAVD 88. The peak water level for system dam failure routing is 26.82 ft (8.17 m) NAVD 88, shown in Figure 2.4.4-6. The system and non-system dam failure simulations considered the 500-year storm in the unsteady flow routing. Therefore, the contribution from failure of all hypothetical dams is approximately 1.07 ft. As per JLD-ISG-2013-01, the still ~~-~~water level resulted from failure of all dams can be calculated by superimposing the net difference from non-critical dams and system dams. ~~The HEC-RAS dam breach and unsteady flow routing models predicted that the peak water level in the Guadalupe River at the LMGS site~~The total still peak water level is the summation of the system and non-system peak water levels, which totals to 27.89 ft (8.50 m) NAVD 88.~~,~~

The maximum wave height out of all fetches was calculated with an $H_{1\%}$ of 5.93 ft (1.81 m). The maximum calculated wind setup is 0.77 ft (0.23 m). The maximum vertical extent of the runup on the embankment on the west of the site was calculated to be 8.58 ft (2.62 m). The accounted SLR is 1.18 ft (0.36 m). ~~without considering the wind wave effects owing to the failure of the Canyon and Coleto Creek dams, would be at elevations 32.5 ft (9.9 m) NAVD 88 and 32.1 ft (9.8 m) NAVD 88, respectively (See Figure 2.4.4-5 and Figure 2.4.4-6). The peak discharges at a distance upstream of the LMGS site would be 1.33x106 cfs (3.77x104 cms) and 1.11x106 cfs (3.14x104 cms), respectively. The flood waves would take more than 23 hr and 1 hr to reach the LMGS site after the failure of Canyon and Coleto Creek Dams, respectively. The simulated maximum dam break water surface profiles from Canyon and Coleto Creek Dams to the downstream boundary of the models are depicted in Figure 2.4.4-5 and Figure 2.4.4-6, respectively.~~

The maximum water surface elevation is 38.42 ft (11.71 m) NAVD 88 with still and wave runup considerations, which is lower than the left riverbank elevation of 41.72 ft (12.71 m) NAVD 88 with a margin of 3.3 ft. Therefore, it is concluded that the LMGS is not impacted by upstream dam failure.

~~The elevation of the left bank of the Guadalupe River at this location is 33.31 ft (10.15 m) NAVD 88, which indicates that the left bank of the river is not overtopped at the site location.~~

2.4.4.2.2 Failure of On-Site Water Control or Storage Structures Analysis

HEC-RAS 2D Version 6.4.1 (USACE, 2023b) was used to evaluate the flooding potential because of the breaching of the on-site Basins #5 and #31. The program solves either the two-dimensional (~~-~~2D) Shallow Water equations (with optional momentum additions for turbulence, wind forces, mud and debris flows, and Coriolis effects) or the 2D Diffusion Wave equations (USACE, 2020). The 2D unsteady flow equations solver uses an Implicit Finite Volume algorithm. The implicit solution algorithm allows for larger computational time steps than explicit methods. The Finite Volume Method provides an increment of improved stability and robustness over traditional finite difference and finite element techniques. The software was designed to use unstructured computational meshes but can also handle structured meshes. Within HEC-RAS, computational cells do not have to have a flat bottom, and cell faces/edges do not have to be a straight line~~,~~ with a single elevation. Instead, each computational cell and cell face is based on the details of the underlying terrain. This type of model is often referred to in the literature as a “high resolution sub-grid model”. The term “sub-grid” means it uses the detailed underlying terrain (sub-grid) to develop the geometric and hydraulic property tables that represent the cells and the cell faces. The HEC-RAS model is approved by the Federal Emergency Management Agency (FEMA) for use in the ~~FIS~~ Flood Insurance Study (FEMA, 2020).

HEC-RAS 2D is capable of simulating water levels and flow rates of the flood waves resulting from a breached section in an embankment (in a 2~~-~~D domain). Obstructions, such as buildings and embankments, can be incorporated into the model.

For simulating flood levels from the breach of the on-site basins, the model domain was delineated in such a way that the entire basins are included, together with the areas surrounding the nuclear islands of LMGS. The extent of the HEC-RAS 2D model is illustrated in Figure 2.4.4-7. The model boundaries are established away from the nuclear island area and safety-related SSCs to prevent boundary conditions from affecting flood levels evaluated within the nuclear island area and to ensure the stability of the model. Boundaries of the model are established through performing sensitivity analysis. In general, the boundary of the model followed the combined West and East Coloma Creek watershed. However, to increase efficiency in the model, the northern and southern extents of the model are limited. The northern extent is established at a location that the basin failure flood does not reach and the southern boundary is established away from the site. From the east, the model naturally flows into the Victoria Barge Canal.

The model covers an area of approximately 18,000 ac (7284 ha), about 4.5 mi (7.2 km) in the east-west direction and about 7.5 mi (12.1 km) in the north-south direction. The numerical grid was generated with RAS Mapper module. The horizontal grid size is 40 by 40 ft (12.1 by 12.1 m) within the entire model domain. The model grid is refined to 10 by 10 ft (3.1 by 3.1 m) at the West Coloma Creek, edges of the embankments, and ditches around the site. The model has 689,222 cells. Figure 2.4.4-8 shows the numerical grid of the on-site basin embankment breach model in the vicinity of the LMGS site. As part of a sensitivity test to demonstrate grid size independence, a coarser grid model was developed with 100 by 100 ft (31 by 31 m) grid sizes, and the simulation results were compared with the finer grid model. The water level results between the two models were not significantly different.

All buildings are modeled as “dry points” by elevating them higher than finish grade by minimum of 10 ft (3.1 m), representing a situation in which the buildings are blocked. Figure 2.4.2-6 presents the site layout.

Assumptions in the On-Site Basin Breach Analysis

In the on-site basin breach analysis, the following assumptions were adopted:

1. Base-flow is not considered for the West and East Coloma Creeks as they are not gaged and do not run with continuous flow. They are designed to transfer storm water from upland agricultural lands to Powderhorn Bay.

2. To develop the most conservative case, no evaporation was considered in the model. This is conservative because storm water lost due to evaporation could reduce inundation depth and period of inundation.

3. Buildings within the watershed are considered to be solid and impervious. Storm water must move around buildings and is not removed from the model by flowing into buildings.

4. Security barrier (Delay Barriers and Jersey Barriers) details are not provided and are not considered in this version of the calculation.

5. Realistic site-specific SCS curve numbers are calculated based on soil type, vegetation, and land use for watershed surfaces and a conservative, yet reasonable, curve number is applied to the watersheds.

6. It is assumed that the surface area of the basin is constant along its depth. This is a conservative assumption that results in maximizing the available volume of water inside basins.

7. Basin failure scenarios are developed based on an event-based approach assuming a reasonably realistic combination of worst-case basin failure modes and initial pool elevations (FEMA, 2013).

8. Due to lack of information regarding an emergency spillway for the basins, it was conservatively assumed that the water level can rise to the top of the basin and overflow during a hydrologic event.

9. The failure and removal of the breached section in the embankments would be instantaneous.

10. All internal dikes within the basin would also fail and be removed instantaneously, coinciding with the breaching of the entire basin.

Topography and~~&~~ Bathymetry

The 2018 USGS LiDAR data for South Texas (USGS, 2018) with nominal pulse spacing of 2.3 ft (0.70 m) and vertical accuracy of 8 in. (5.8 cm) at a 95 percent confidence level are used in this analysis. The data were developed based on a horizontal projection/datum of North American Datum of 1983 (2011), UTM Zone 14, meters, and vertical datum of NAVD 88 (GEOID12B), meters. Figure 2.4.4-9 and Figure 2.4.4-10 show the model representation of the bathymetry for the entire model and for the nuclear island area where the safety-related SSCs are located, respectively. Finish site grade is set at 31 ft (9.45 m) NAVD 88.

Boundary Conditions

The model domain is bounded by four boundaries. The northern and eastern boundaries were positioned far enough so that the maximum flood level at the LMGS safety-related SSCs due to operating basin breach would occur before the flood wave front reaches the two boundaries. The southern and western boundaries are set to normal flow with 0.001 slope.

Initial Conditions & Basin Failure Hydrographs

In accordance with ANSI/ANS 2.8, 1992 (ANSI/ANS, 1992), the initial water level in the reservoir should be set to the level that corresponds to one half of the local probable maximum precipitation. According to site drawings, the normal operating water level of the basins is 34.5 ft (10.5 m) NAVD 88. The local intense precipitation of the site is 55.7 in. (141.5 cm) in 72 hr. This corresponds to elevation of 36.8 ft (11.2 m) NAVD 88, which is 1.2 ft (0.37 m) lower than the top of the embankments. Conservatively, the top of embankment is used for the analysis during a hydrologic event and 37 ft (11.3 m) NAVD 88 is used during sunny day failure.

The initial flow velocities in the model domain were all set to zero.

Because Froehlich's 2008 equation (Froehlich, 2008) only provides the peak discharge and formation time, a simple triangular shape hydrograph is fit to each scenario as presented in Figure 2.4.4-11.

Model Parameter Selection

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:

- USDA NRCS Technical Release 55 (TR-55) (USDA-SCS, 1986)
- HEC-RAS Reference Manual (USACE, 2020)
- Open-Channel Hydraulics (Chow, ~~1959~~1988)

The references listed above provide a range of acceptable values for Manning's roughness coefficients (see Table 2.4.4-2) for overland floodplain flow. Because flow is generally expected to be directed away from the nuclear island area, a higher Manning's roughness coefficient generally results in higher water levels.

Based on conterminous U.S. land cover (USGS, 2021), the model domain is mainly covered with Cultivated Crops and Hay/Pasture. Therefore, a Manning's n value of 0.05 is applied to the model.

The SCS curve number method is a standard, widely used, and efficient method for determining the amount of runoff from a breach scenario. A combination of land use and hydrologic soil group is used to determine curve number. The dominant soil group and land use are agricultural and “D,” respectively. The corresponding curve number to such a combination is 87 (USDA-SCS, 1986). A curve number of 90 is conservatively applied to the entire applicable watershed. A curve number of 99 is assigned to the site to account for paving.

Flood Levels from Failure of On-Site Water Control or Storage Structures

The finished floor grade of the nuclear island is at elevation 31.5 ft (9.6 m) NAVD 88, which is 6.5 ft (2.0 m) below the crest elevation of Basins #5 and #31. Two scenarios are considered with Basin #5 breach location varying along its northern embankment.

Scouring and sedimentation are expected due to high velocity near the breached portion of the basins and also from the dike itself. To evaluate the impact of sedimentation on site flooding, the topography of the site and nearby areas were carefully reviewed. Sedimentation inside West Coloma Creek is not a reasonable consideration as the creek is protected by dikes on two sides that prevent eroded sediment from finding its way inside the creek. Sedimentation inside ditches around the basin and LMGS site is not a plausible scenario as velocity is still high in those areas. It is expected that sedimentation would occur downstream of the site, aligned with the main direction of the flow, and where the velocity is low.

A velocity of 1 ft/s (0.31 m/s) was considered as the threshold and the area shown in Figure 2.4.4-12 was identified as the proper sedimentation region for sensitivity run. This area is on the west side of Coloma Creek floodplain and about 2000 ft (609.6 m) south of the site. Conservatively, the entire floodplain is raised by 2 ft (0.61 m) to simulate blockage by sedimentation. Both scenarios are then run with this modified grade to evaluate the impact on the site. It should be noted that this consideration is conservative as there are incised basins on the west side of the site that can capture the majority of the scouring as deposition inside them in case of any basin failure.

Modeling results are presented in Figure 2.4.4-13 through Figure 2.4.4-20. Under Scenario 1, the maximum inundation at the LMGS site varies from 5 to 10 in. (12.7 to 25.4 cm) with the western edges of the site experiencing flooding depth to the range of 1 ft (2.54 cm) (see Figure 2.4.4-15).

Under Scenario 2, maximum inundation at the LMGS site varies from 6 to 12 in. (15.2 to 30.48 cm) with the western edges of the site experiencing flooding to the range of 1.3 ft (0.4 m) (see Figure 2.4.4-19). However, Scenario 2 is mainly impacting the southern part of the site compared with Scenario 1, which impacts both southern and northern parts of the site.

Velocity at the site reaches 6 ft/s (1.8 ~~meters per second [~~m/s~~]~~) with the edges of the site experiencing velocity to the range of 10 ft/s (3.1 m/s), which indicates proper sizing of the protection for the site berm should be performed (see Figure 2.4.4-16 and Figure 2.4.4-20).

Roads that are inundated by more than 1 ft (0.31 m) are identified for both scenarios (see Figure 2.4.4-13 and Figure 2.4.4-17).

Sedimentation sensitivity showed that the deposition downstream of the site does not impact the inundation depth at the LMGS site.

2.4.4.3 Water Level at the Plant Site

Analyses of the dam failures within the Guadalupe River basin and the failure of the on-site basin embankments showed that the critical flood level at the LMGS site is controlled by the basin embankments failure. The safety-related SSCs within the nuclear island are affected by potential dam failures as explained below.

2.4.4.3.1 Water Level at the Plant Site from the Failures of Upstream Dams

In accordance with the guidelines in ANSI/ANS-2.8-1992 (ANSI/ANS, 1992), the maximum upstream dam breach flood level at the plant site must consider the effects of wind setup and wave run-up from the coincidental occurrence of a 2-year design wind event. ~~The most conservative wind setup and wave run-up estimated for the probable maximum tsunami is adopted here. As discussed in Section 2.4.6, t~~The estimated wind setup is 0.77 ft (0.23 m) and wave runup is 8.58 ft (2.62 m)~~8.2 ft (2.5 m)~~. The accounted SLR is 1.18 ft (0.36 m). The maximum ~~PMF~~ upstream dam breach 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 27.89~~32.5~~ ft (8.50~~9.91~~ m) NAVD 88. This results in maximum dam failure water level at the LMGS site to be at elevation 38.42~~40.7~~ ft (11.71~~12.41~~ m), 3.3 ft lower than the left riverbank elevation of 41.72 ft (12.71 m) NAVD 88. Therefore, it is concluded that the LMGS is not impacted by upstream dam failure.

~~. Therefore, the left bank of the river is overtopped by wind-wave runup effect. The site is not flooded during a Guadalupe River dam failure event as the overtopped spillage is 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 areas surrounding the site.~~

~~2.4.4.3.3~~2.4.4.3.2 Water Level at the Plant Site from the Failure of On-Site Water Control or Storage Structures

The predicted maximum water level at the nuclear island and the LMGS site for the postulated breaching of the operating basin northern embankments is at elevation 32.5 ft (9.91 m) NAVD 88. The effects of wind setup and wave run-up are not considered as the result of the short duration of the flooding incident.

This indicates that the safety-related SSCs are flooded by maximum of 1 ft (0.31 m) during on-site basin failures.

Flood protection for the safety-related SSCs is needed and is discussed in detail in Section 2.4.10.

~~2.4.4.3.5~~2.4.4.3.3 Sedimentation and Erosion

An upstream dam failure event would not flood the LMGS site, as is evidenced by the model prediction, which shows that the total maximum ~~still~~ water level of elevation 38.42 ft (11.71 m) ~~32.5 ft (9.91 m)~~ NAVD 88 for this event is lower than the left bank of the Guadalupe River, which is ~~of~~ about 41.72~~33.91~~ ft (12.71~~10.34~~ m) in the LMGS nuclear island vicinity. As such, the LMGS would not be affected by a upstream dam failure event; therefore,~~area that would be inundated by the flood flow is a considerable distance away from the LMGS nuclear island. Therefore,~~ no sedimentation or erosion affecting the safety functions of LMGS is expected.

Figure 2.4.4-14, Figure 2.4.4-16, Figure 2.4.4-18, and Figure 2.4.4-20 show the predicted flood flow velocity pattern at the LMGS site for two on-site scenarios. The maximum flow velocity is estimated to be approximately 11 ft/s (3.35 m/s) in localized areas near the nuclear island, and much lower-3 ft/s (0.9 m/s) per second or less-in the general vicinity. In addition, this flood flow velocity would not be sustained because the flood level would decline in response to the continuous draining of the basins. Under such flow conditions, erosion or scouring is not expected to affect the nuclear island areas and safety functions of LMGS. However, protection from localized high velocities on the periphery of the raised nuclear island is required.

Potential breaching of the operating basin embankments would produce sediments that could be carried with the flood flow. The source of these sediments would be the basin/reservoir embankment material and nearby local bed scours. However, given the range of the estimated flood flow velocity from the embankment breaching scenarios, it is expected that the coarser material and debris would likely settle out within a short distance from the breaching locations, and the finer material would be carried along in the flood flow past the nuclear island areas. With the subsidence of the breaching flood, more material would be settled out inside and out outside the nuclear island areas. But the impact of sedimentation the required safety functions of the safety-related SSCs is not a concern as these SSCs are protected inside the buildings.

~~Flood protection for the safety-related SSCs is needed and will be discussed in detail in Section 2.4.10.~~

References

2.4.4-1 ANSI/ANS, 1992. Determining Design Basis Flooding at Nuclear Power Reactor Sites, ANSI/ANS 2.8-1992, American National Standards Institute/American Nuclear Society, July 1992. (Historical technical reference)

2.4.4-2 CEDAS-ACES Version 4.03. Veri-Tech, Vicksburg, MS.

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~~Table 2.4.4-1~~
~~Breach Parameters for Canyon and Coleto Creek Dams~~

~~Breach Parameters~~	~~Canyon Dam~~	~~Coleto Creek Dam~~
~~Average width of breach (ft.)~~	~~1120~~	~~408~~
~~Breach bottom elevation (ft., MSL)~~	~~750~~	~~55~~
~~Top of embankment (ft., MSL)~~	~~974~~	~~120~~
~~Lake water level (ft., MSL)~~	~~977~~	~~123~~
~~Side slope of breach~~	~~1:1~~	~~0~~
~~Breach time to failure (hr.)~~	~~0.1~~	~~0.1~~

~~Note: MSL data is equivalent to NGVD 29 and is assumed to be the same as NAVD 88 as the result of a very small difference.~~

Table 2.4.4-1
Dam Failure Inflow Junction Connection from HEC-HMS to HEC-RAS

HEC-HMS Hydrograph-Element Name	HEC-RAS Station Number
Canyon Lake	294.428
Guad_S130	294.246
Guad_S140	286.033
BearCr_S010	285.88
Guad_S142	271.721
Comal_abv_Guad	271.611
Guad_R190	252.376
GeronimoCk_abv_Guad	240.451
Guad_S164	239.743
Guad_S166	236.945
Guad_S168	215.727
Guad_R228	172.723
SanMarcos_S050	169.547
SanMarcos_R050	169.347
Guad_S210	144.264
PeachCr_S040	143.528
PeachCr_R040	142.353
Guad_S220	114.503
McCoyCr_S010	114.124
SandiesCr_R030	100.923
Guad_S230	100.784
SandiesCr_S040	100.467
Guad_S240	84.032
Guad_R295	48.073
ColetoCk_abv_Guadalupe	32.069
San Antonio 500-year Discharge	6.04

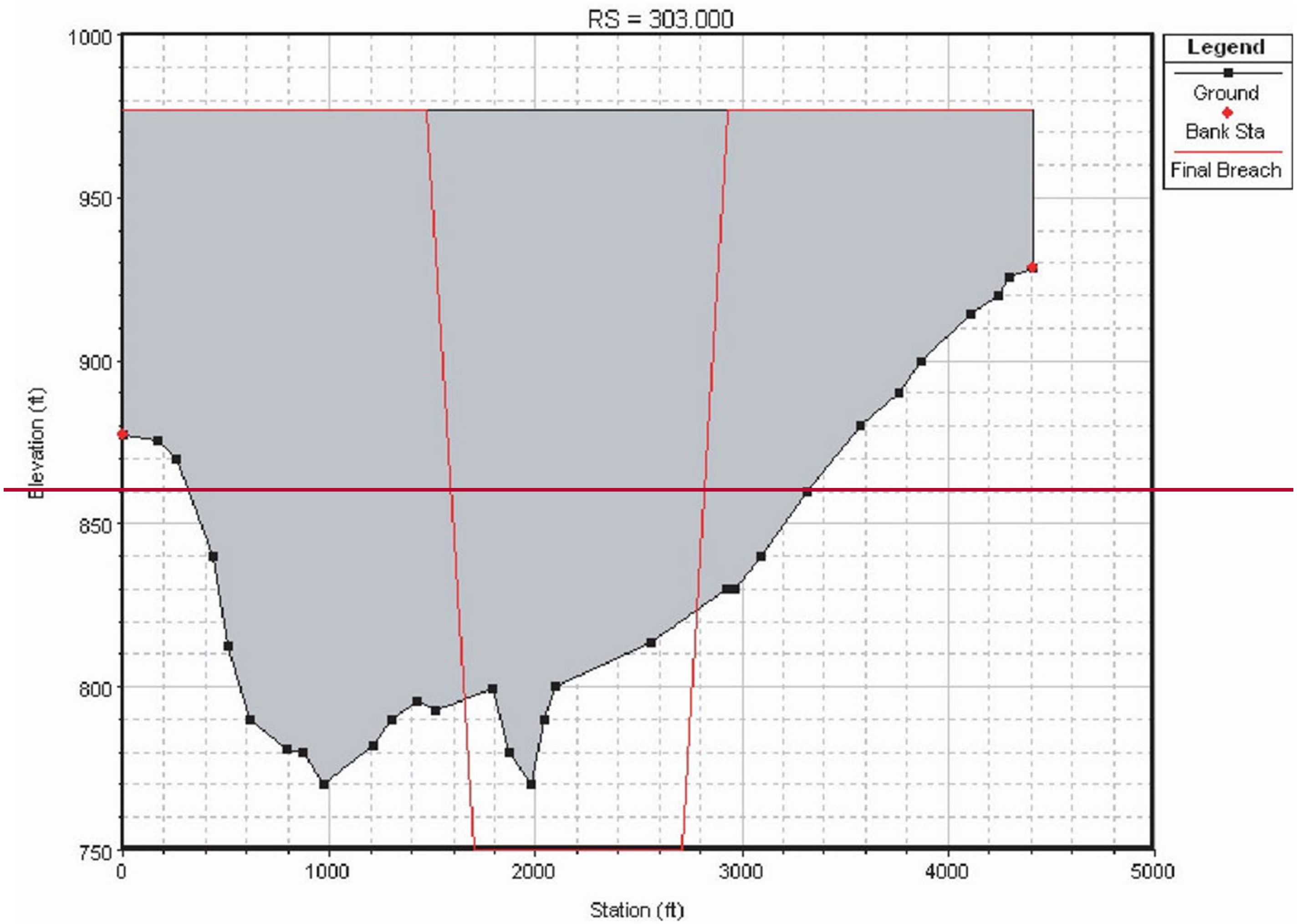
Table 2.4.4-2
Manning's Roughness Coefficients

Manning's Roughness Category[a]	Manning's Roughness Coefficient		
	High	Medium	Low
Cultivated Crop	0.05	0.035	0.02
Hay/Pasture/ Grassland	0.05	0.027	0.025
Pavement and Buildings	0.05	0.03	0.011
Water	0.05	0.038	0.025

a) References USDA-SCS, 1986; USACE, 2020; Chow, ~~1959~~1988

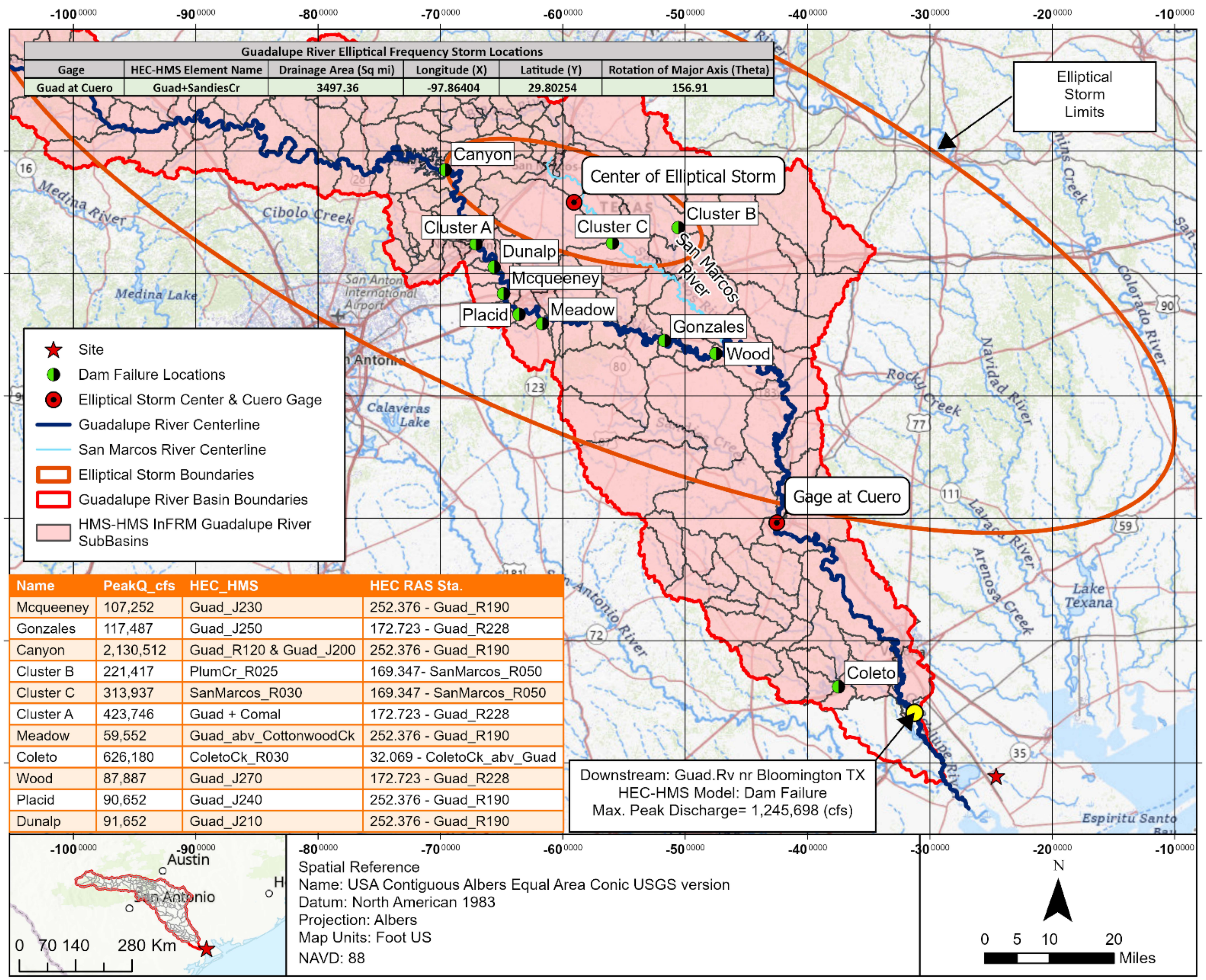
Figure 2.4.4-1
On-Site Operating Basins

~~Figure 2.4.4-2~~
~~Model Cross Section at Canyon Dam~~



Note: The legend "Bank Sta" represents the limit of the channel left and right banks in the model.

Figure 2.4.4-2
Dam Failure & 500-Year Elliptical Storm with Outfall Gage Junction at Cuero



Guadalupe River Elliptical Frequency Storm Locations					
Gage	HEC-HMS Element Name	Drainage Area (Sq mi)	Longitude (X)	Latitude (Y)	Rotation of Major Axis (Theta)
Guad at Cuero	Guad+SandiesCr	3497.36	-97.86404	29.80254	156.91

Name	PeakQ_cfs	HEC_HMS	HEC RAS Sta.
Mcqueeney	107,252	Guad_J230	252.376 - Guad_R190
Gonzales	117,487	Guad_J250	172.723 - Guad_R228
Canyon	2,130,512	Guad_R120 & Guad_J200	252.376 - Guad_R190
Cluster B	221,417	PlumCr_R025	169.347- SanMarcos_R050
Cluster C	313,937	SanMarcos_R030	169.347 - SanMarcos_R050
Cluster A	423,746	Guad + Comal	172.723 - Guad_R228
Meadow	59,552	Guad_abv_CottonwoodCk	252.376 - Guad_R190
Coleto	626,180	ColetoCk_R030	32.069 - ColetoCk_abv_Guad
Wood	87,887	Guad_J270	172.723 - Guad_R228
Placid	90,652	Guad_J240	252.376 - Guad_R190
Dunalp	91,652	Guad_J210	252.376 - Guad_R190

~~Figure 2.4.4-3~~
~~Model Cross Section at Coleto Creek Dam~~

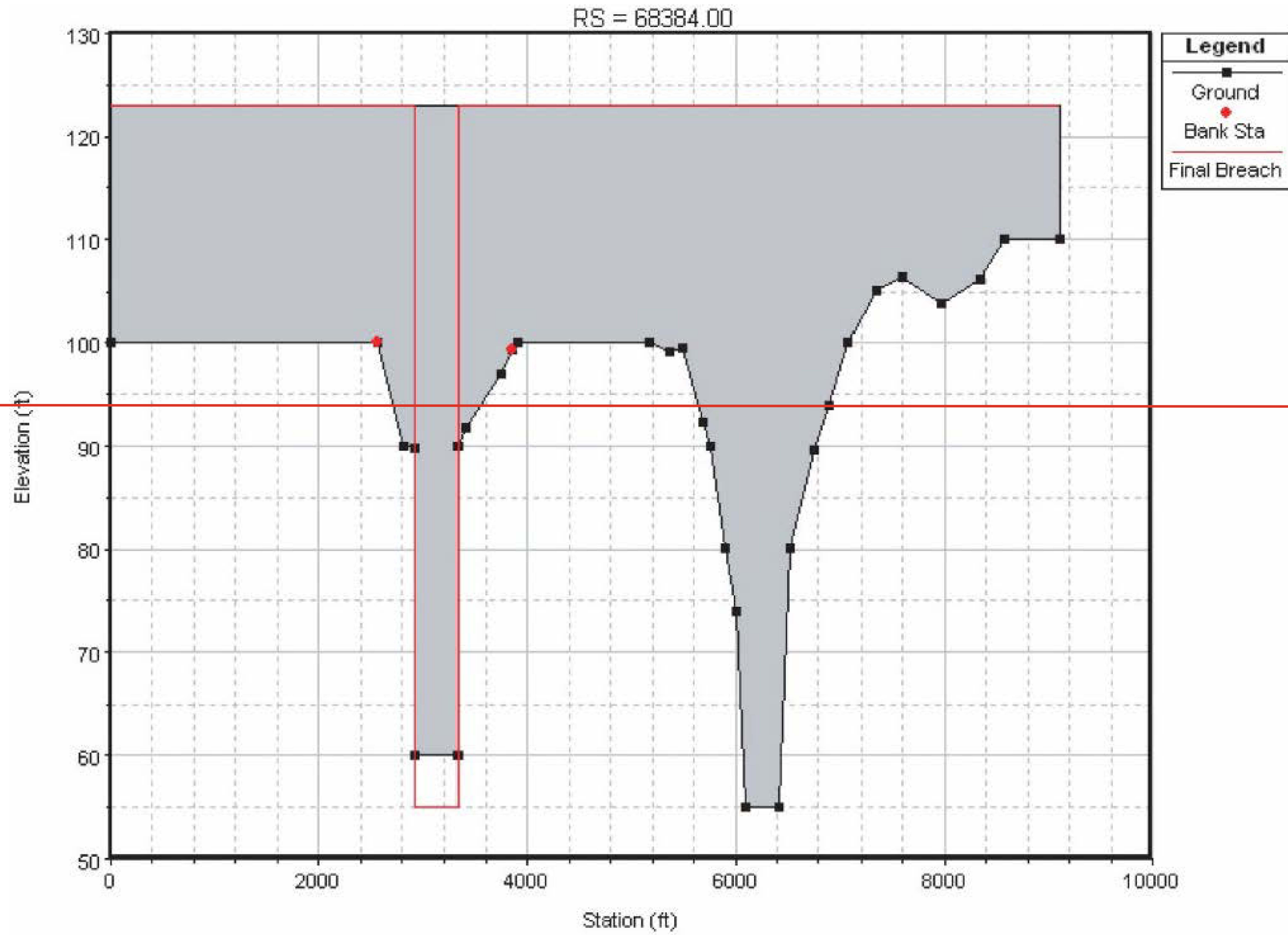
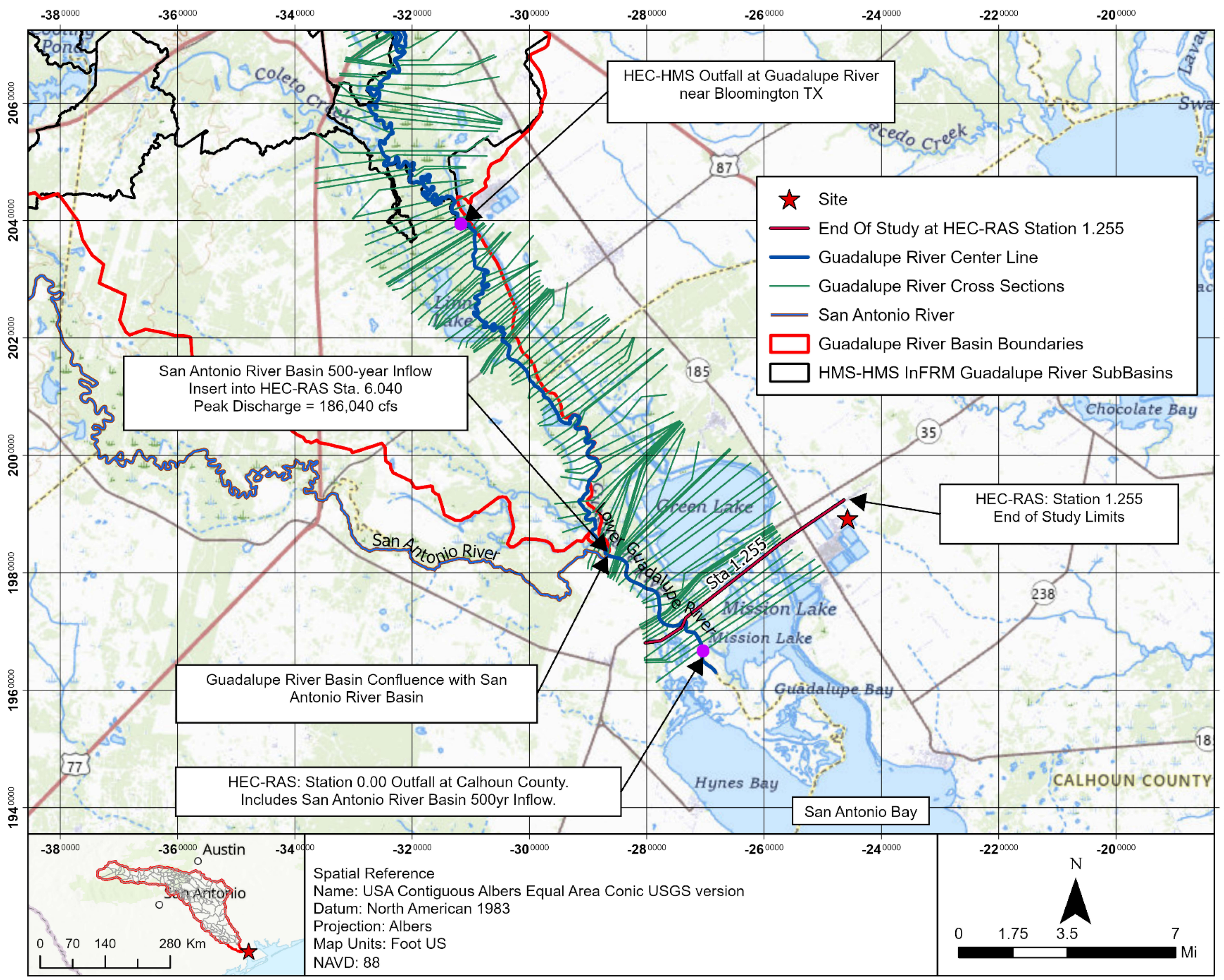


Figure 2.4.4-3
HEC-RAS Study Limit



~~**Figure 2.4.4-4**~~
~~**Plan View of Cross Section Locations on the Guadalupe River**~~

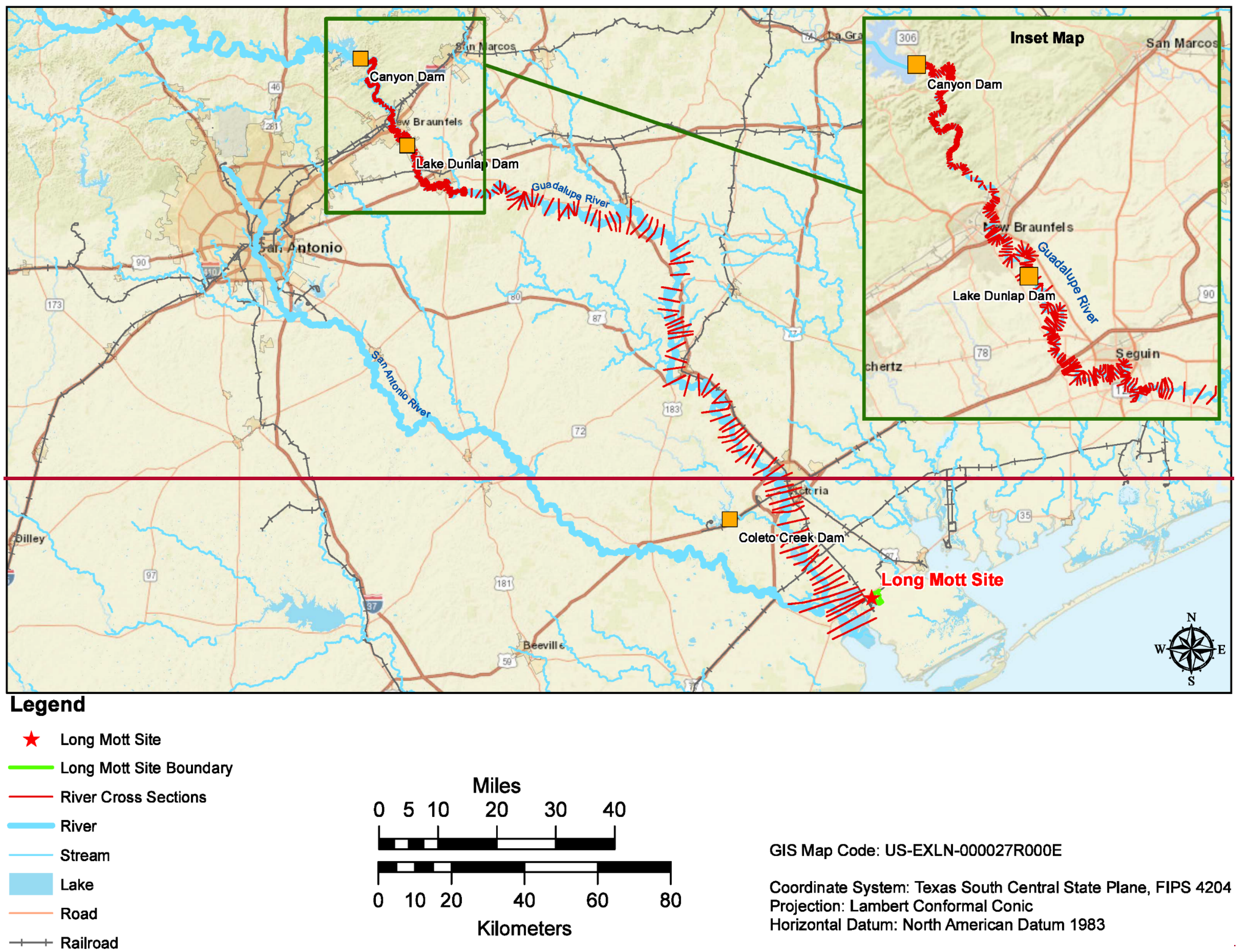
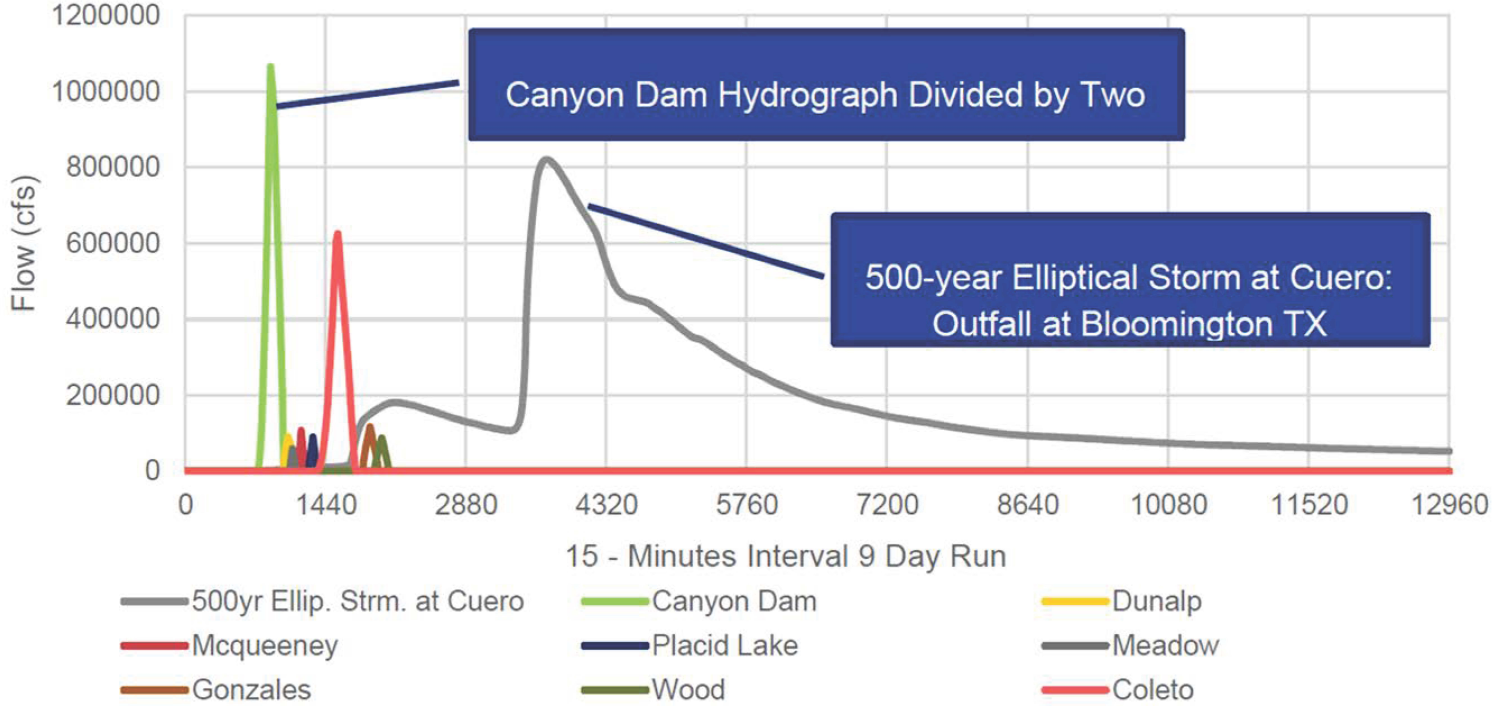
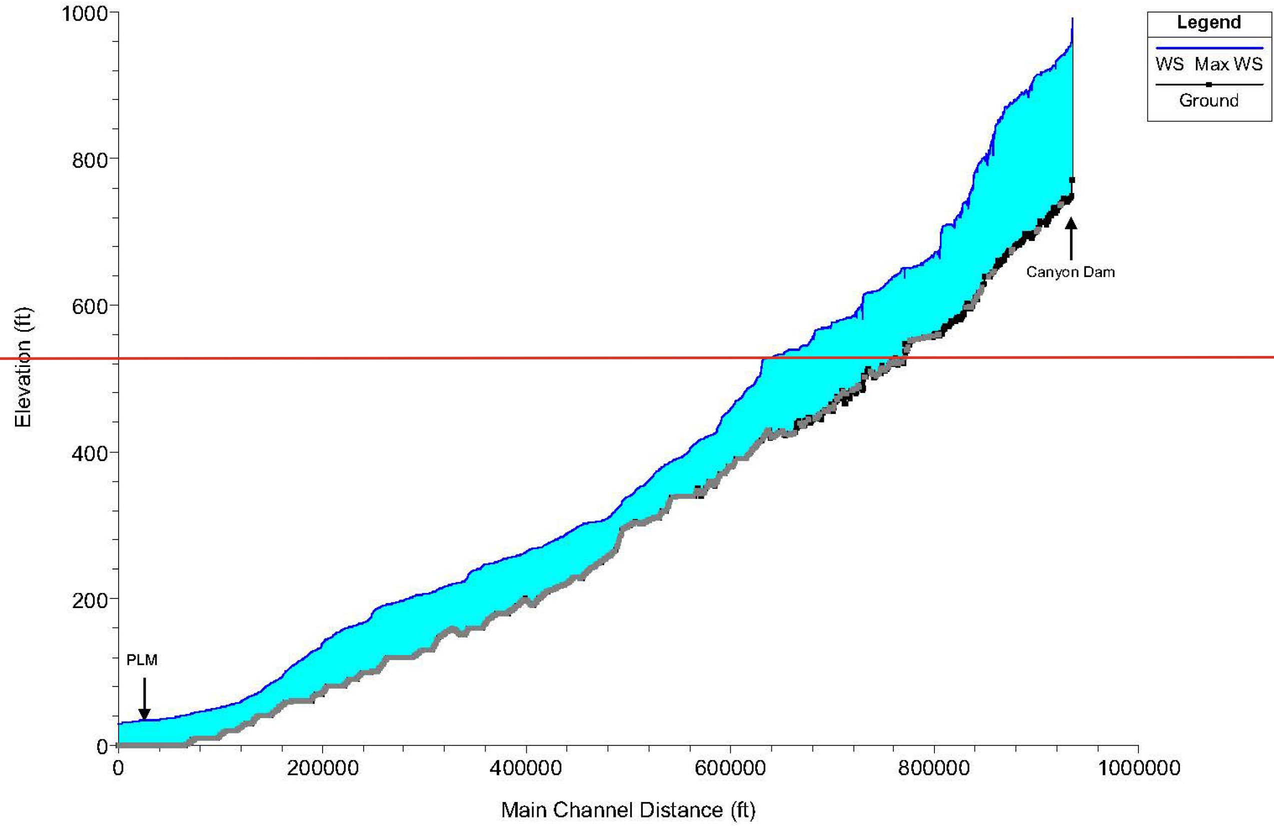


Figure 2.4.4-4
HEC-HMS Guadalupe River Dam Failure Hydrographs Case 1

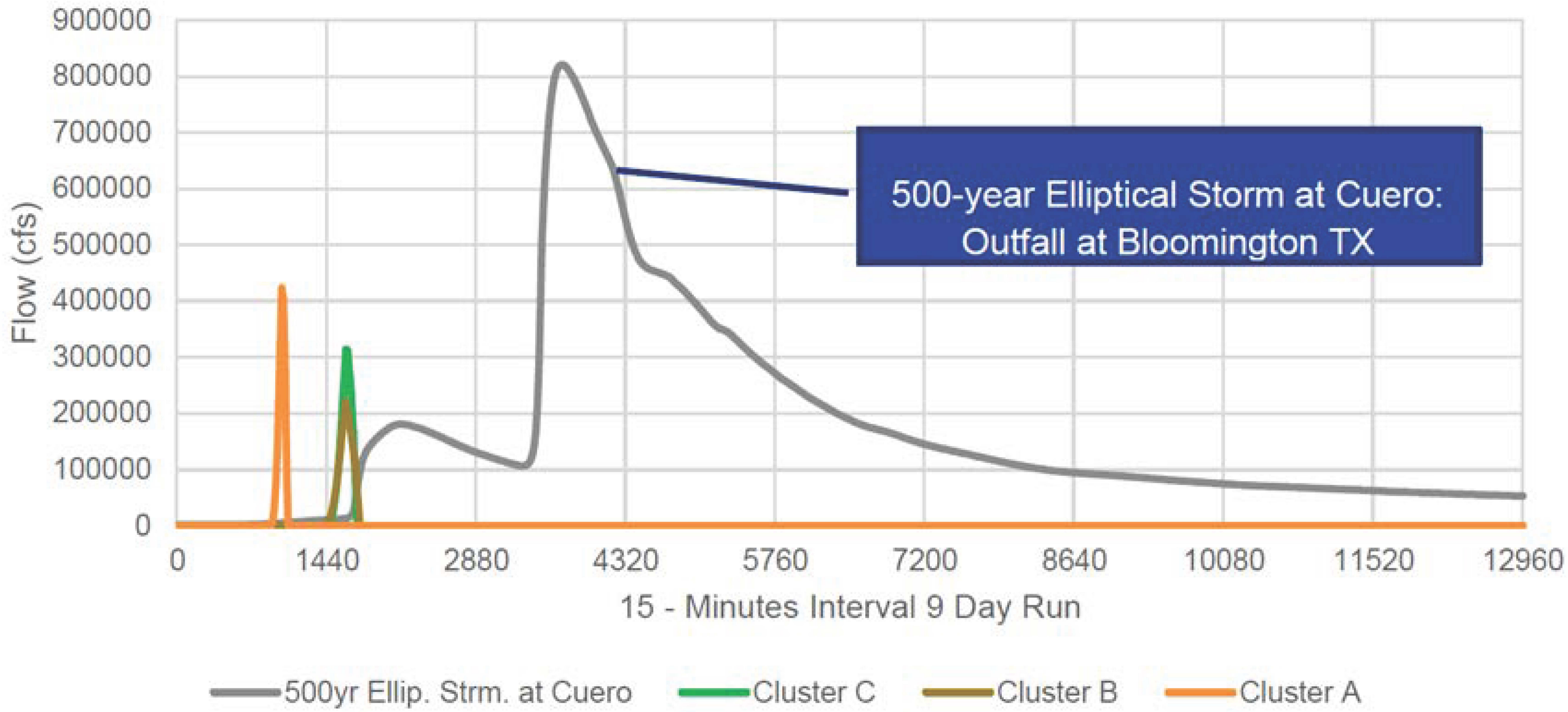


~~Figure 2.4.4-5~~
~~Maximum Water Level Profile (NAVD 88) for the Canyon Dam Breach Case~~

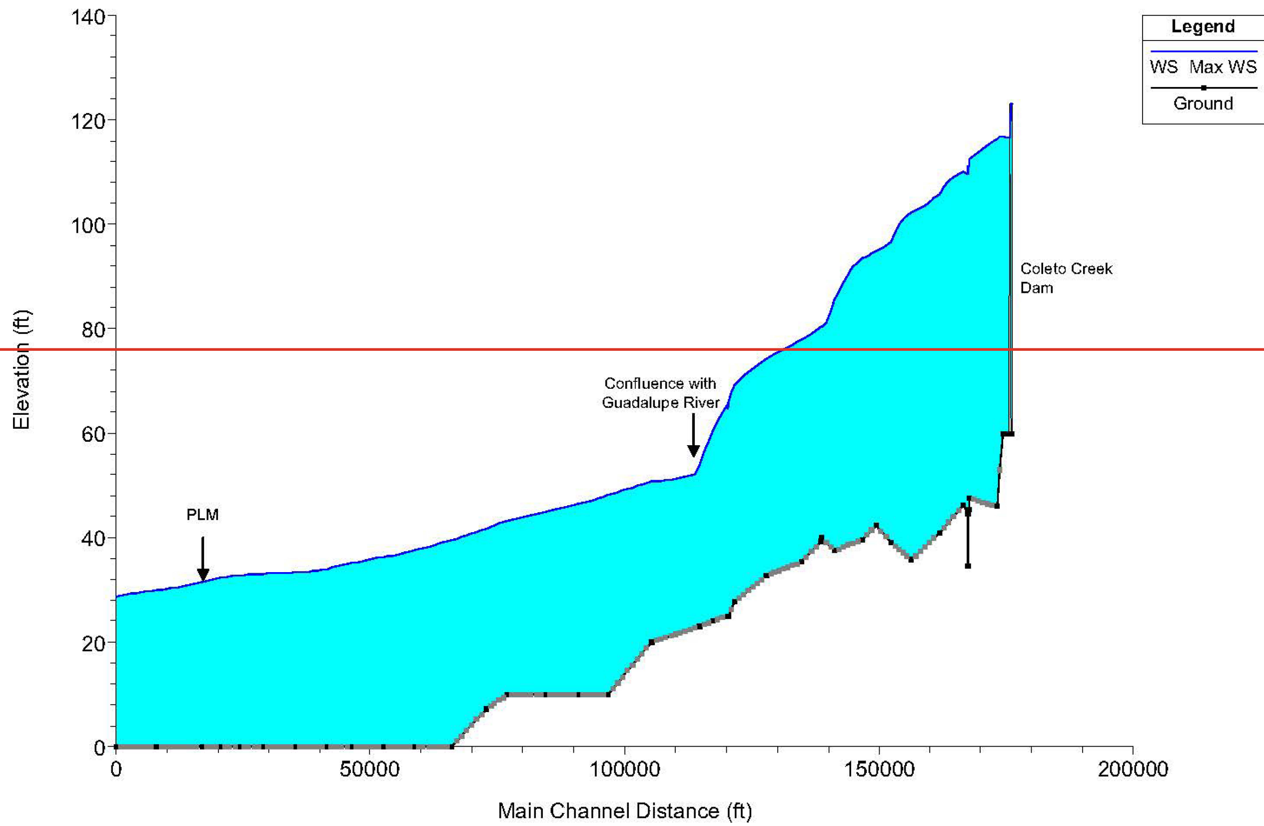


Note: The legend “WS Max WS” represents maximum water surface level.

Figure 2.4.4-5
HEC-HMS Gudalupe River Dam Failure Hydrographs Case 2



~~Figure 2.4.4-6~~
~~Maximum Water Level Profile (in NAVD 88) for the Coleto Creek Dam Break Case~~



Note: The legend "WS Max WS" represents maximum water surface level

Figure 2.4.4-6
HEC-RAS Study Limit

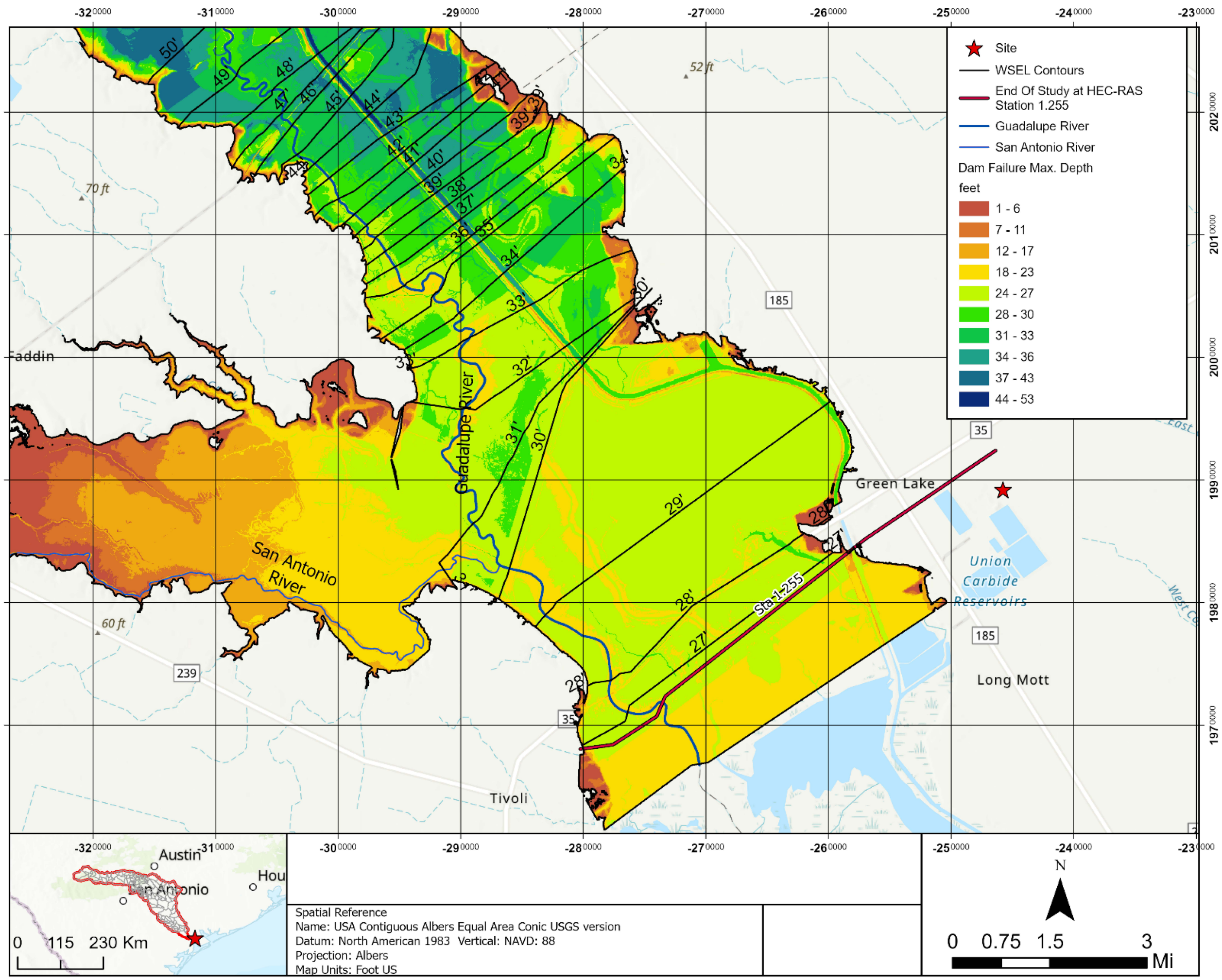


Figure 2.4.4-7
HEC-RAS 2D Model Outline and Boundary Conditions for On-Site Pond Failure Analysis

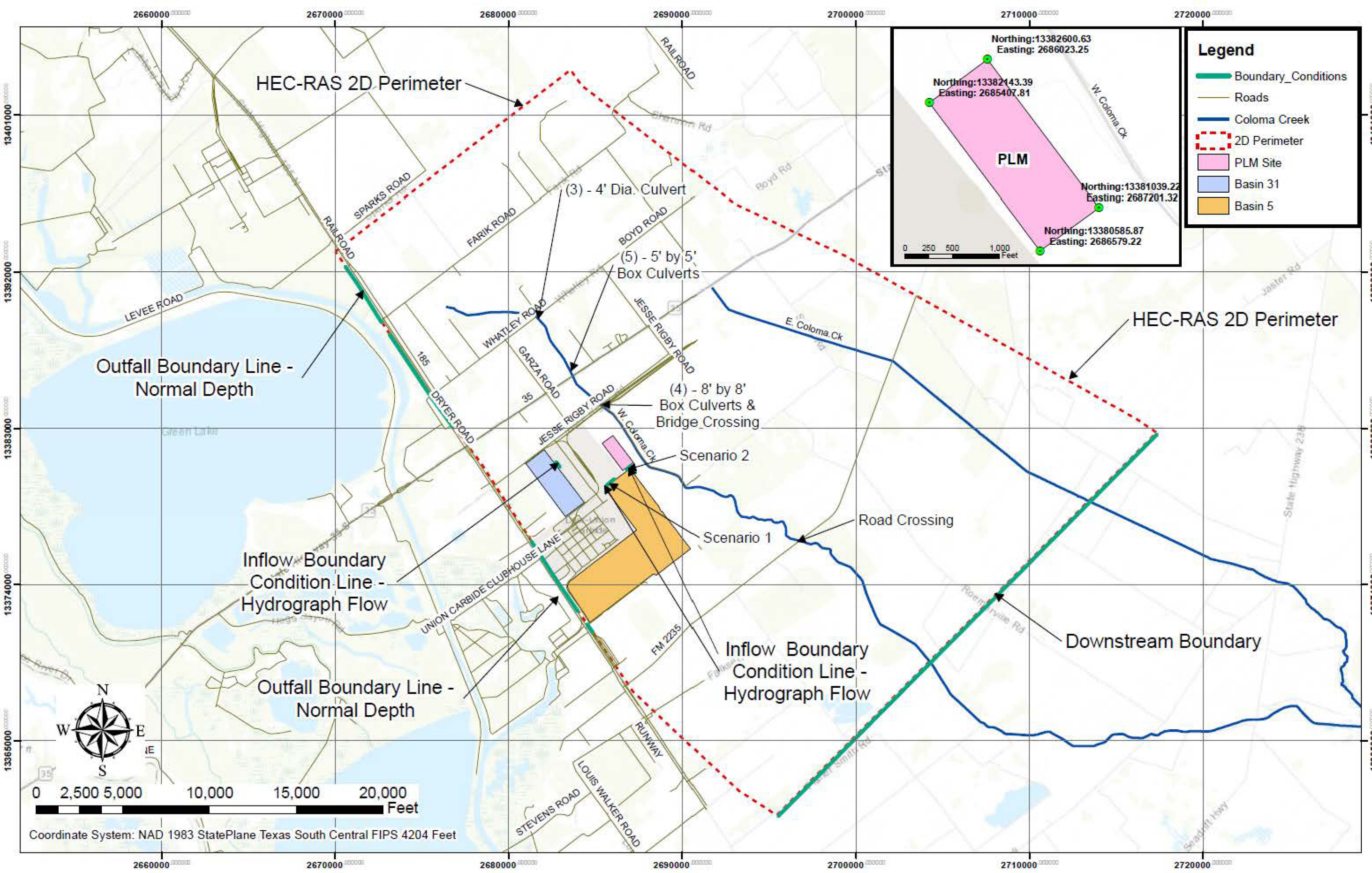


Figure 2.4.4-8
HEC-RAS 2D Model Grid in the Vicinity of the Long Mott Generating Station Site

Figure 2.4.4-9
Model Topography/Bathymetry

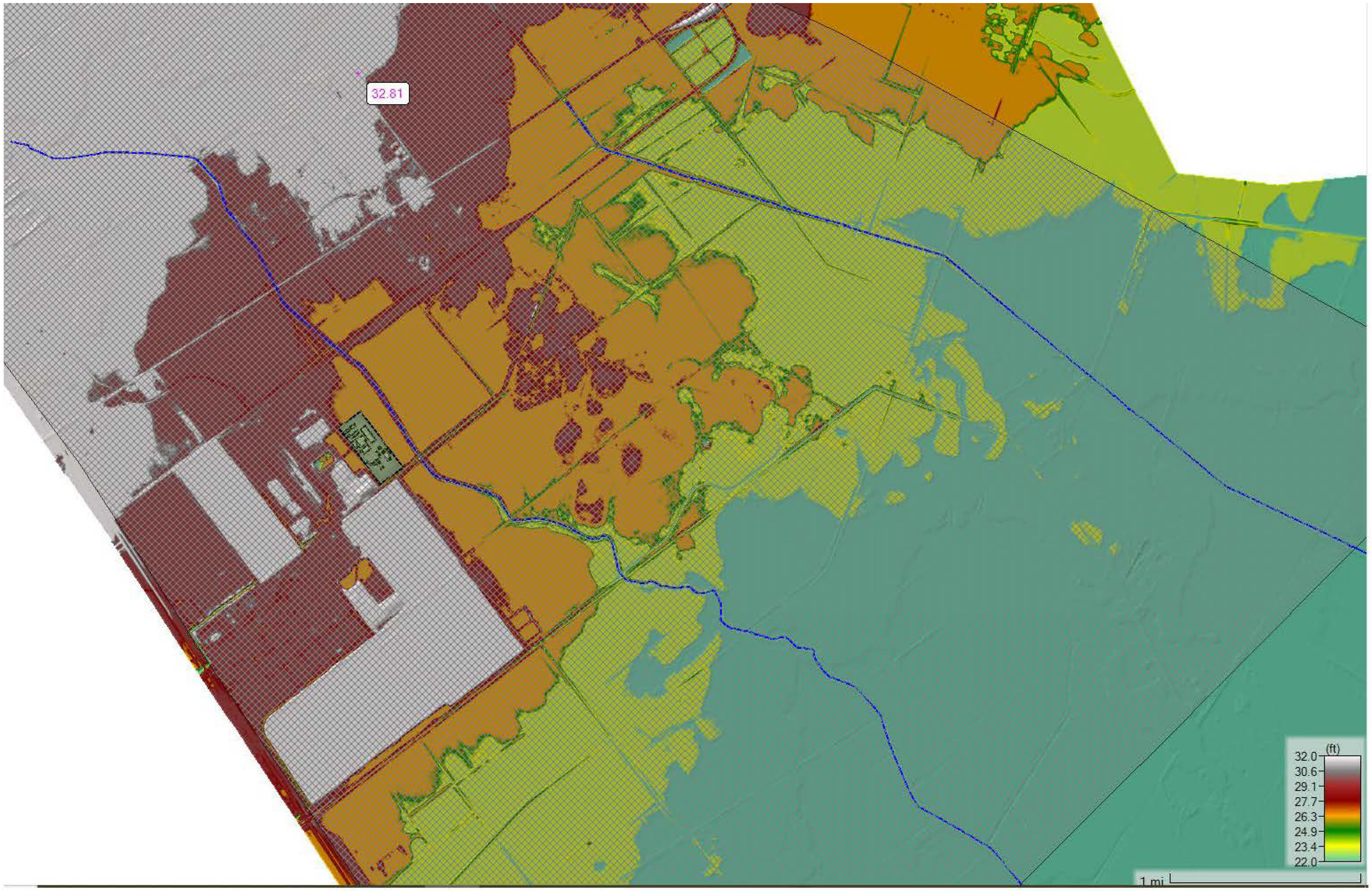


Figure 2.4.4-10
Model Topography/Bathymetry Focused

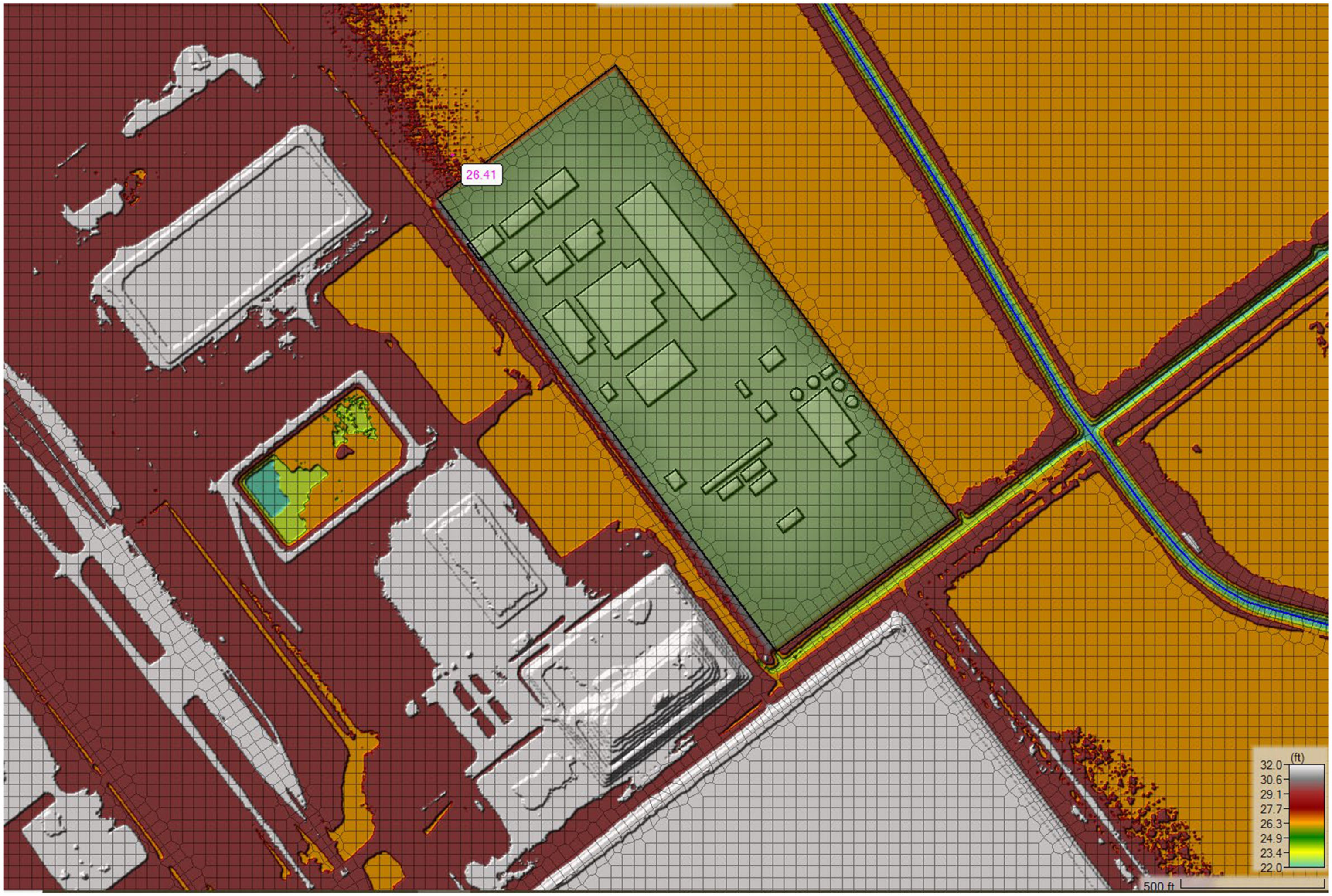


Figure 2.4.4-11
Breach Hydrographs

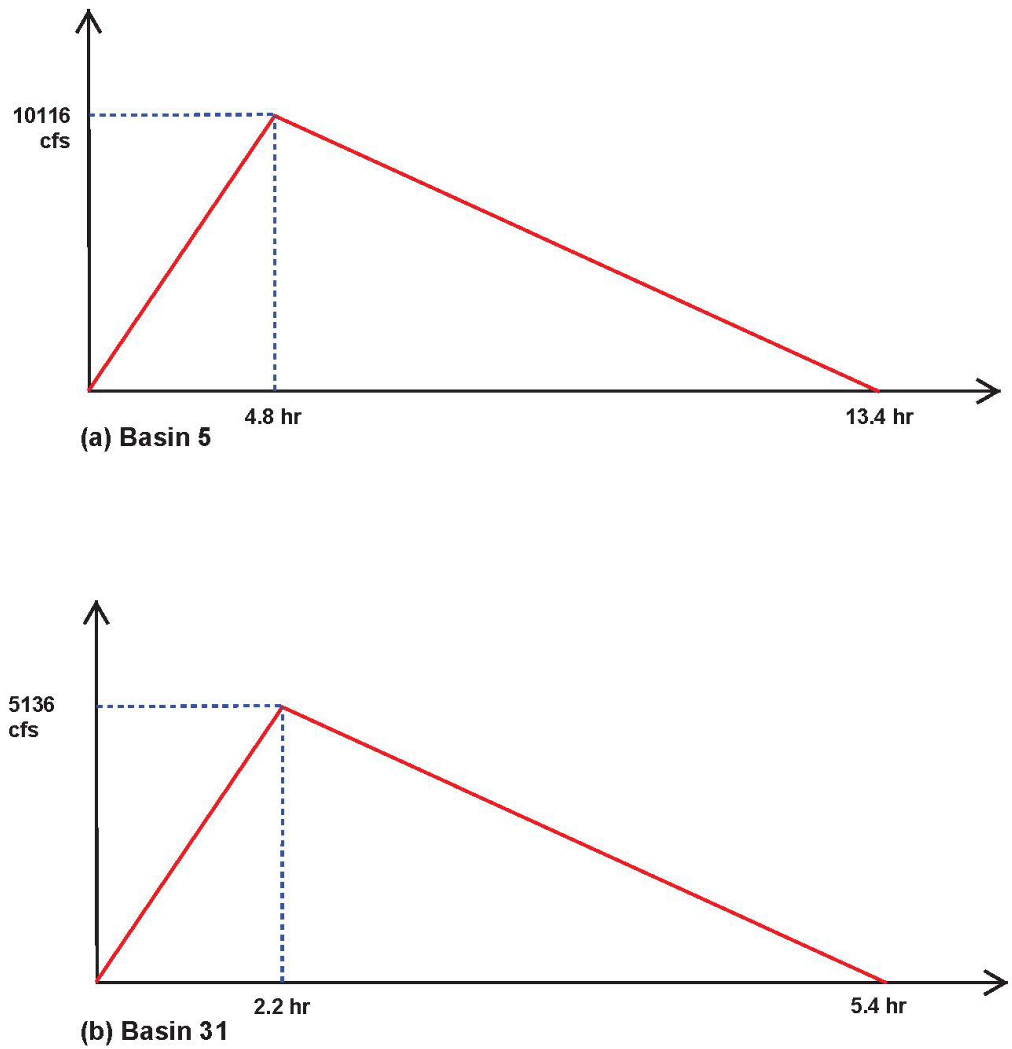


Figure 2.4.4-12
HEC-RAS 2D Sedimentation Region

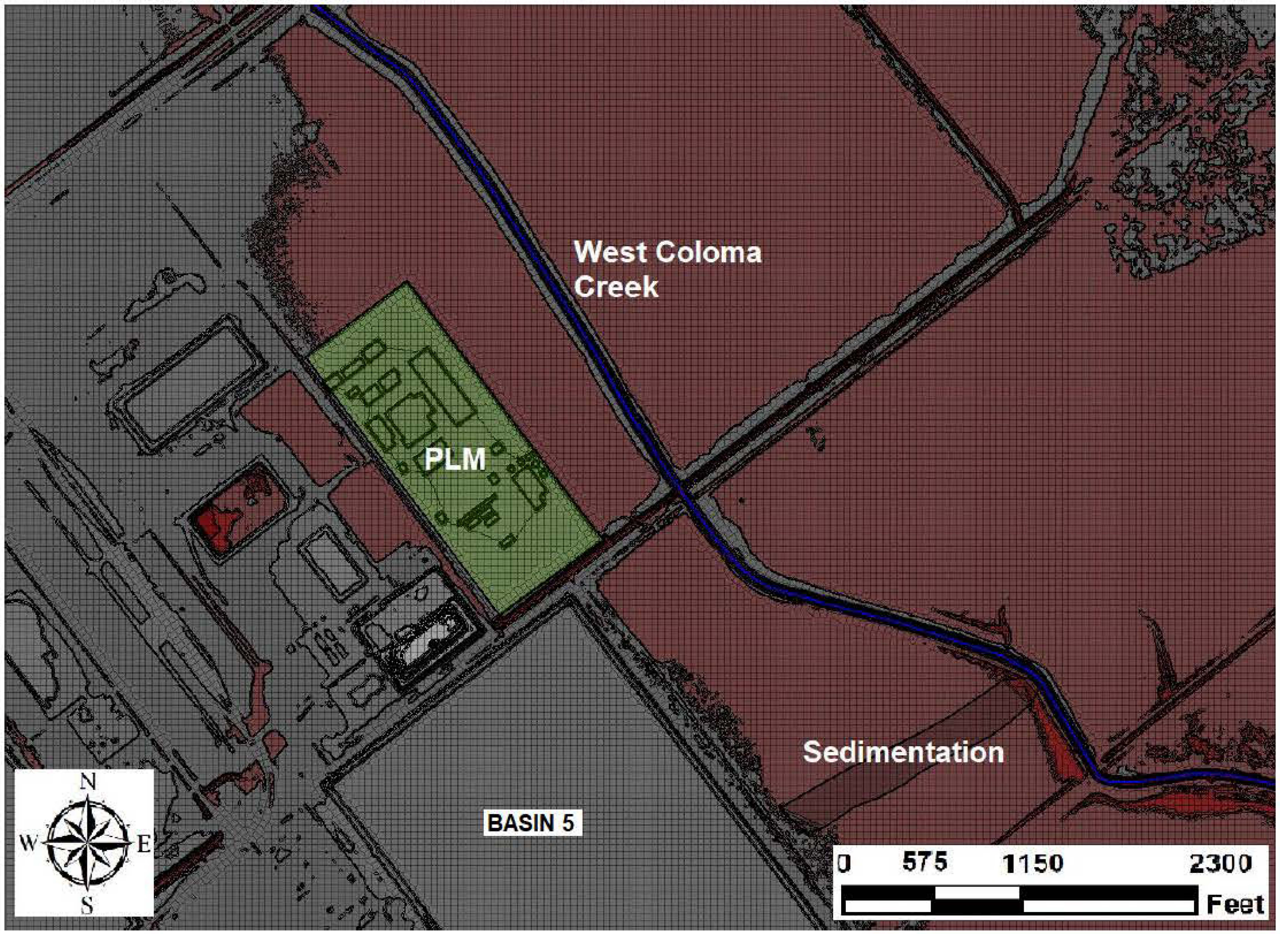


Figure 2.4.4-13
Pond Breach Scenario 1 – Overall Inundation Depth

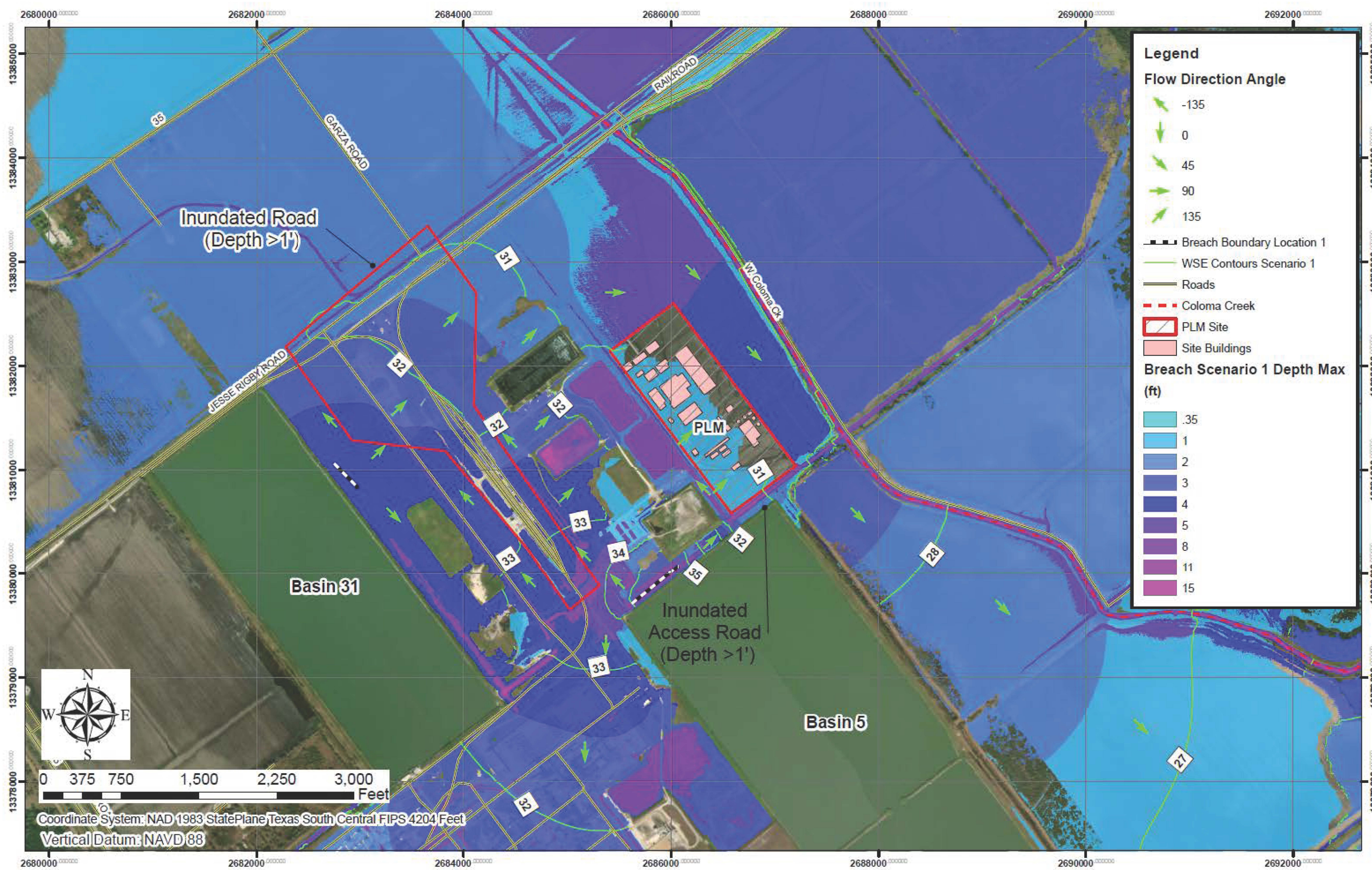


Figure 2.4.4-14
Pond Breach Scenario 1 – Overall Velocity

Figure 2.4.4-15
Pond Breach Scenario 1 – Flood Depth at the Long Mott Generating Station Site

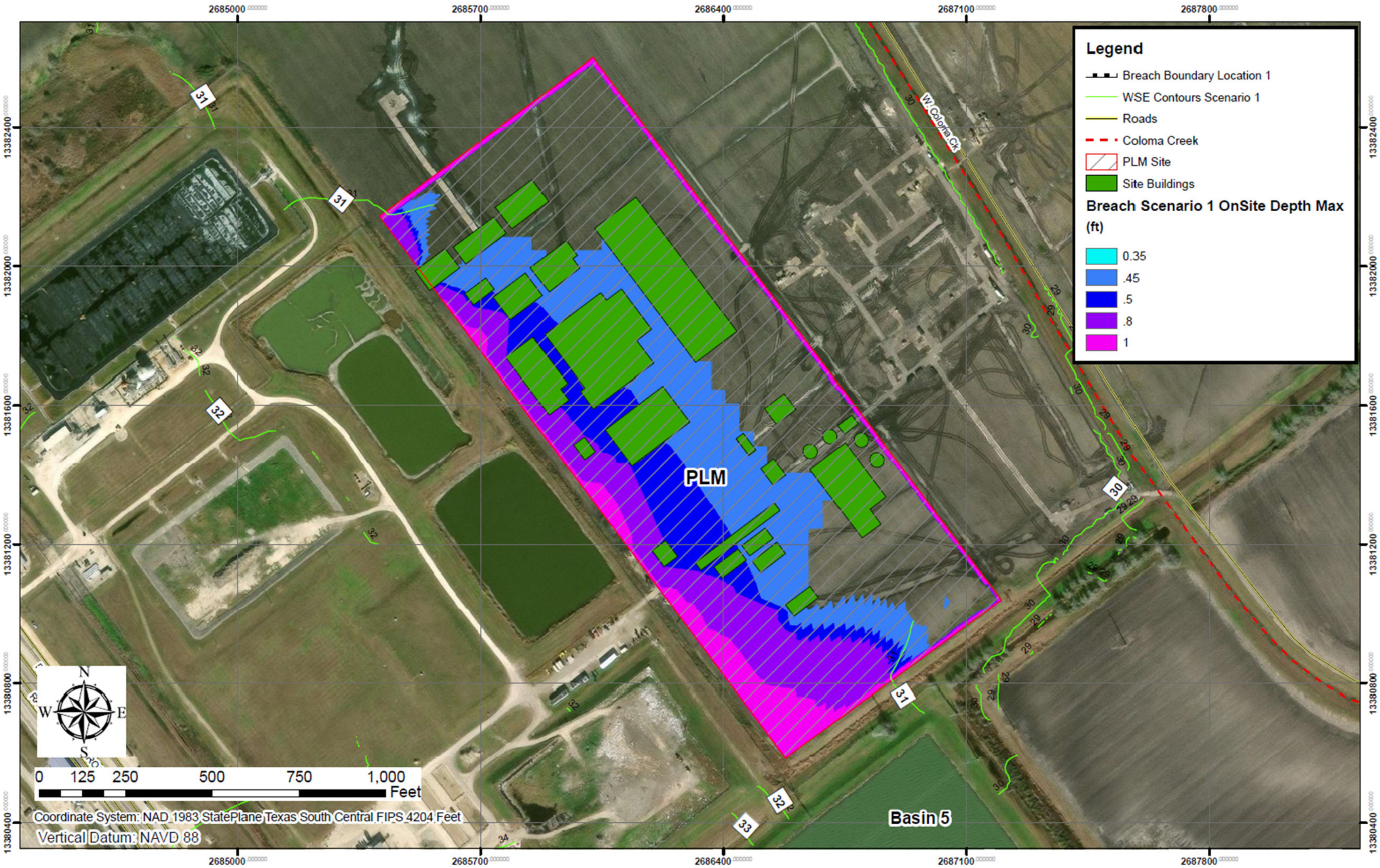


Figure 2.4.4-16
Pond Breach Scenario 1 – Flood Velocity at the Long Mott Generating Station Site

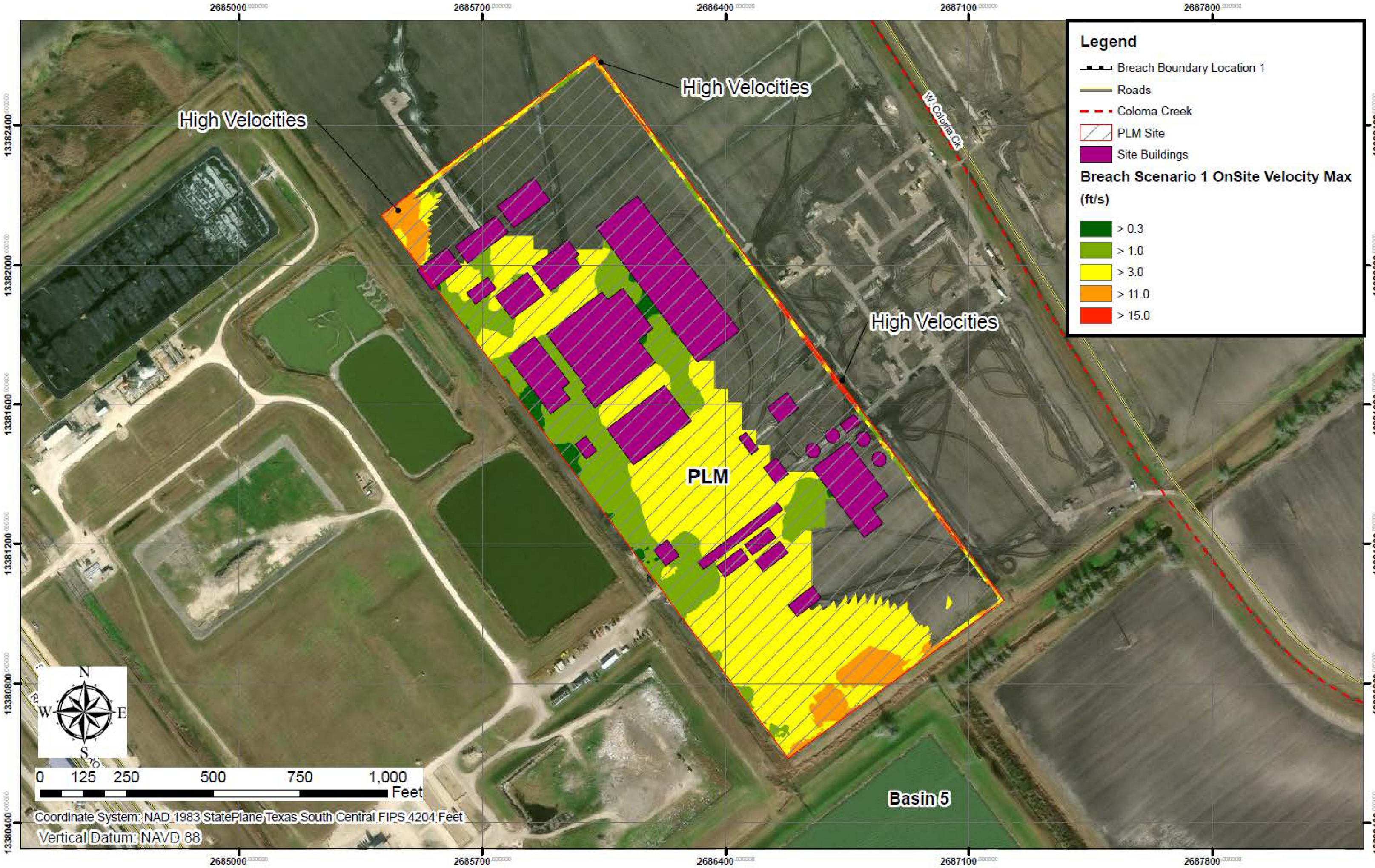


Figure 2.4.4-17
Pond Breach Scenario 2 – Overall Inundation Depth

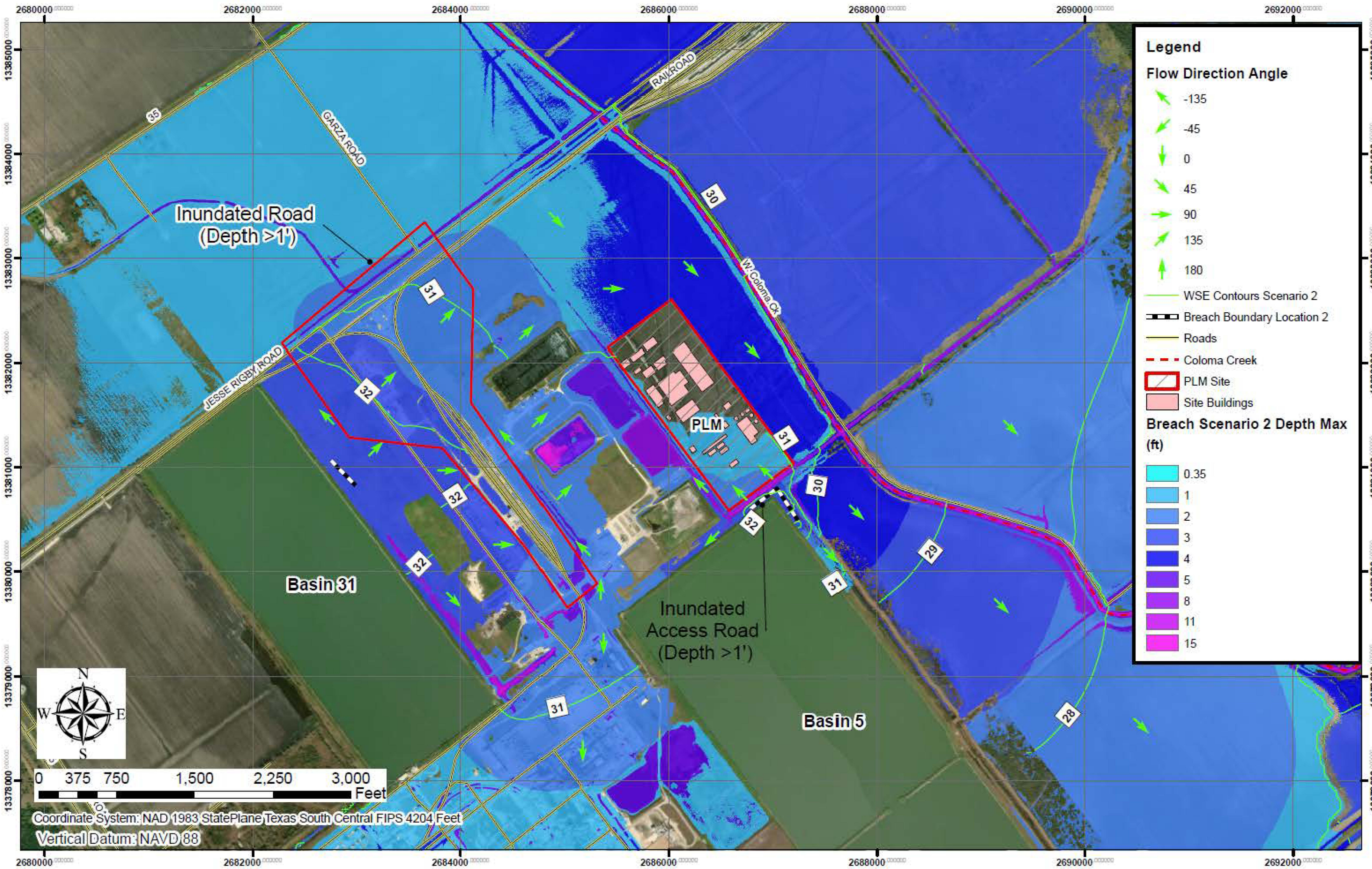


Figure 2.4.4-18
Pond Breach Scenario 2 – Overall Velocity

Figure 2.4.4-19
Pond Breach Scenario 2 – Flood Depth at the Long Mott Generating Station Site

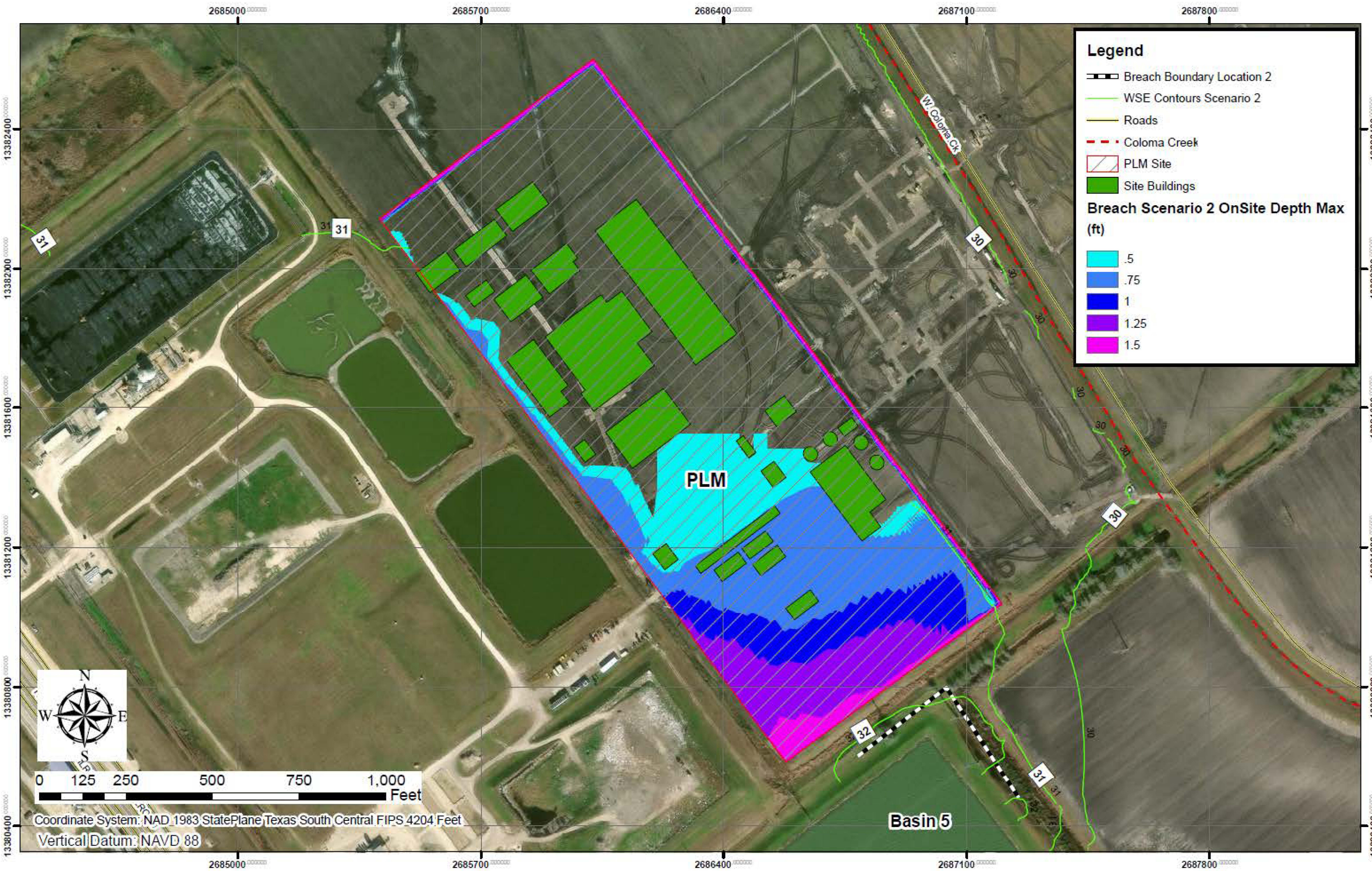


Figure 2.4.4-20
Pond Breach Scenario 2 – Flood Velocity at the Long Mott Generating Station Site