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### 2.4.4 Potential Dam Failures

This subsection evaluates the potential flooding hazards to the power block as a result of potential dam failures. As described below, the maximum flooding levels at the VCS site resulting from the worst-case scenarios of upstream dam failure and postulated cooling basin breach were estimated to be 68.4 feet (20.8 meters) NAVD 88 and 91.0 feet (27.7 meters) NAVD 88, respectively, which are below the minimum finished site grade of 95.0 feet (29.0 meters) NAVD 88 at the power block of VCS.

The VCS site is located on the west side of the Guadalupe River in Victoria County, Texas, about 36 river miles (60 kilometers) upstream of the Gulf of Mexico shoreline. There are a total of 29 dams with storage capacity in excess of 3000 acre-feet (AF) ($3.7\times10^{-3}$ cubic kilometer) on the Guadalupe River and its tributaries upstream of the VCS site. These dams and reservoirs are owned and operated by different entities including the Guadalupe-Blanco River Authority (GBRA), U.S. Army Corps of Engineers (USACE), and Natural Resources Conservation Service (previously the United States Department of Agriculture, Soil Conservation Service). Figure 2.4.4-1 shows the locations of the 29 dams. Specific information on these dams that is relevant to the flood risk assessment of the VCS site is summarized in Table 2.4.4-1 based on data collected from the Texas Commission on 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 (Reference 2.4.4-1), are subject to periodic re-evaluation in consideration of continuing downstream development. Hydrologic criteria contained in Rule §299.14 of Title 30 (Table 3), Hydrologic Criteria for Dams, is 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.

Two dam failure scenarios that are most critical to flooding at the VCS site are considered. They are: (1) the postulated cascade failure of the major upstream dams on the Guadalupe River basin, and (2) the breaching of the embankments of the onsite cooling basin that is located south of the power block (all orientations are given with respect to the plant's north; i.e., 50 degrees counterclockwise of the true north). These two scenarios form the basis of the maximum flood level evaluation for VCS resulting from potential dam failures. There are no new dams planned in the Guadalupe River basin in the next 50 years, according to the 2007 Texas water plan (Reference 2.4.4-2). Dam failure scenarios and postulated flood risk are discussed in the following subsections.

Because there are no impoundments on the Guadalupe River downstream of VCS that are used to supply water to the safety-related facilities, no failure analysis of downstream dams was conducted. The Lower Guadalupe diversion dam and salt water barrier downstream of the VCS site is used to restrict salt water 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 facilities of the plant.

Data used in preparing this subsection was gathered from different sources that adopted different elevation datums. Because the elevation differences between mean sea level (MSL) (the same as National Geodetic Vertical Datum of 1929 [NGVD 29]) and NAVD 88 are small at Canyon and Coleto Creek Dams (Reference 2.4.4-15), no conversion was made between the datums.

### 2.4.4.1 Dam Failure Permutations

#### 2.4.4.1.1 Failures of Upstream Dams in the Guadalupe River Basin

Of all the dams on the Guadalupe River upstream of the VCS site, the Canyon Dam would generate the most significant dam break flood risk on the site. Canyon Dam has the largest dam height of 224 feet (68.3 meters) and the largest reservoir storage capacity of about 1.2 million acre-feet (MAF) (1.48 $km^3$) at its peak. The Coleto Creek Dam (on Coleto Creek, which is a tributary of the Guadalupe River), located about 21 river miles (34 km) upstream, is closest to the VCS site among all major upstream dams. The Coleto Creek Dam has a height of 65 feet (19.8 meters) and a top-of-dam storage capacity of 0.17 MAF (0.21 $km^3$).

Two separate failure scenarios are postulated for the upstream dams:

Scenario No. 1: 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 Nuclear Society, ANSI/ANS-2.8-1992 (Reference 2.4.4-3).

Scenario No. 2: Failure of Coleto Creek Dam induced by a non-hydrologic event. The failure is to occur coincidentally with a 500-year flood or a one-half PMF on Coleto Creek, in accordance with ANSI/ANS-2.8-1992 (Reference 2.4.4-3).

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 4.5 percent of that of Canyon Dam (Table 2.4.4-1).

The remaining 21 “off-channel” dams are located on the tributaries of the Guadalupe River between Canyon Dam and the VCS site. These off-channel storage dams were also assumed to have no

effect on the maximum dam break flood level at the VCS site, 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 will not be the controlling scenario in the evaluation of the maximum flood risk at the VCS site. This is because Canyon and Coleto Creek Dams (References 2.4.4-16 and 2.4.4-17, 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 Texas Administrative Code (Reference 2.4.4-1).

#### 2.4.4.1.2 Postulated Failure of the Cooling Basin Embankments

The cooling basin is located south of the power block of the plant. The basin is enclosed by a rolled-earthen embankment that rises an average of 22 feet (6.7 meters) and 37 feet (11.3 meters) above the natural ground at the northern and southern locations, respectively. The centerline of the north embankment is at least 1000 feet (300 meters) south of the power block area (Figure 2.4.4-16). The natural grade between the northern and southern embankments varies from about elevation 80 feet (24.4 meters) to about 65 feet (19.8 meters) NAVD 88, respectively, and the top of the embankment is at elevation 102.0 feet (31.1 meters) NAVD 88, except at piping and spillway crossings where the roadway needs to be elevated. The design pool level of the cooling basin is 90.5 feet (27.6 meters) NAVD 88. The normal maximum operating level of 91.5 feet (27.9 meters) NAVD 88, which includes 1 foot of operating range above the design pool level, is conservatively used in the embankment breach assessment. The normal maximum operating level is about 11.5 feet (3.5 meters) higher than the natural grade near the northern embankment. 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 cooling basin embankment 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 embankment would fail. In this conservative postulation, an embankment section several hundred feet long would translate downstream a short distance from its original location. Further, the postulated breach width was increased incrementally, up to 2034 feet (620 meters), to capture the most critical flooding impact to the site. A 1345-foot (410-meter) or wider breach was found to produce the highest flood level of elevation 90.6 feet (27.6 meters) NAVD 88, which was predicted to occur at the southwest corner of the security wall, an elevated barrier at the security perimeter around the VCS (see Figures 2.4.4-16 and 2.4.4-17).

#### 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 VCS site, is highly improbable because of the wide and flat floodplains and a mild side slope of the river valley.

The potential for waterborne missiles reaching the VCS site as the result of an upstream dam failure is not critical because the site is located above the flood plain of the Guadalupe River, and no flood flow is expected at the site. There is also no potential for waterborne missiles from debris as the result of the embankment breach of the cooling basin, because the maximum flood level would be below the minimum finished site grade of 95.0 feet (29.0 meters) NAVD 88 at the power block.

### 2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

#### 2.4.4.2.1 Guadalupe River Basin Dams

The dams on the Guadalupe River are discussed in Subsection 2.4.4.1. Table 2.4.4-1 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 AF ($3.7\times10^{-3}$ km$^3$) each. Of these 29 dams, Canyon and Coleto Creek Dams were selected for inclusion in the dam break analysis. Dams with less than 3000 AF ($3.7\times10^{-3}$ km$^3$) 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. The top-of-dam storage volume of Canyon Dam is about 1.2 MAF (1.48 km$^3$), based on the elevation-storage capacity curves given in Reference 2.4.4-4. Similarly, the top-of-dam storage volume of Coleto Creek Dam is estimated to be about 0.17 MAF (0.21 km$^3$). The combined maximum capacity of the remaining 27 dams upstream of the VCS site is about 0.19 MAF (0.23 km$^3$).

##### 2.4.4.2.1.1 Conceptual Unsteady Flow Model

The dam breach option of the USACE Hydrologic Engineering Centers River Analysis System (HEC-RAS) Version 3.1.3 (Reference 2.4.4-5) was used to simulate the dam breach flood waves, which were then routed downstream to the VCS site 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.

In the conceptual dam break flood model of Canyon Dam, the dam would fail with the reservoir water level 3 feet (0.9 meters) above the dam crest, at elevation 977.0 feet (297.8 meters) NAVD 88, thereby releasing the flood storage. In accordance with the combined events requirements specified in ANSI/ANS-2.8-1992 (Reference 2.4.4-3), 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-year flood or one-half PMF, whichever is less. In this analysis, a constant flood flow of 571,300 cfs (16,177 cubic

meter per second [cms]) was conservatively used to represent the coincidental flow. This value corresponds to the flood peak at the VCS site as the result of PMF conditions for Canyon Dam and is greater than the one-half PMF at the VCS site (561,650 cfs or 15,904 cms). Details about the PMF values are discussed in Subsection 2.4.3.

The flood wave from the breaching of Canyon Dam would propagate down to the VCS site. The six Guadalupe River dams/reservoirs located between Canyon Dam and the VCS site (Lake Meadow, Lake Placid, Lake McQueeney, Lake Dunlap, Lake Gonzales, and Lake Wood) have a combined maximum storage of about 52,000 AF (0.064 $km^3$). These dams were not assumed to fail in the dam break model because their combined total storage amounts to only about 4.5 percent of the total dam break flood volume at Canyon Dam and would have little impact on the maximum flood level at VCS.

In the conceptual dam break flood model of Coleto Creek Dam, the dam would fail with the water level 3 feet (0.9 meters) above the dam crest, at elevation 123.0 feet (37.5 meters) NAVD 88, thereby releasing the flood storage. In accordance with the combined events requirements specified in ANSI/ANS-2.8-1992 (Reference 2.4.4-3), the evaluation of potential flood risks as a result of non-hydrologic dam failures should also consider a coincidental event equal to a 500-year flood or one-half PMF, whichever is less. In this 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 the VCS site (described in Subsection 2.4.3).

#### 2.4.4.2.1.2 Physical Dam Data and Estimates of Breached Sections

Canyon Dam is an earth dam, located about 275 river miles (443 km) upstream of the VCS site, is 6830 feet (2082 meters) long and 224 feet (68.3 meters) high. Following the guidelines from the Federal Energy Regulatory Commission (FERC) on dam break analysis (Reference 2.4.4-6), five times the dam height (1120 feet or 341.4 meters) 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 feet (273 meters) and 1344 feet (410 meters), respectively. The time to complete the breach was assumed to be 0.1 hour, based on guidelines from the FERC for the estimation of the dam breach parameter (Reference 2.4.4-6). The model cross section for Canyon Dam is shown in Figure 2.4.4-2.

Coleto Creek Dam, an earth dam 18,950 feet (5776 meters) long and 65 feet (19.8 meters) high, is located about 21 river miles (34 kilometers) upstream of the VCS site. The concrete spillway structure has a gross width of 328 feet (100.0 meters) and a total width of 408 feet (124 meters), including the extensions on both sides into the earthen embankment. Following the FERC guidelines (Reference 2.4.4-6), the entire spillway concrete structure is postulated to fail. This is greater than

fives times the dam height that would fail along the earthen embankment (325 feet or 99 meters). The time to complete the breach was also assumed to be 0.1 hour. The model cross section for Coleto Creek Dam is shown in Figure 2.4.4-3.

Table 2.4.4-2 lists the dam breach characteristics used to model the failure of these two dams.

2.4.4.2.1.3 Channel Geometry

For the Guadalupe River, cross section data was obtained from two sources: (1) HEC-RAS models developed for the Federal Emergency Management Agency's (FEMA's) Flood Insurance Study (FIS) along the river (References 2.4.4-7, 2.4.4-8, and 2.4.4-9), and (2) U.S. Geological Survey (USGS) National Elevation Dataset (NED) (Reference 2.4.4-10). The cross section data from FEMA's FIS is composed of three reaches:

a. Comal County, just downstream of Canyon Dam to the River Road Second Crossing in New Braunfels, Texas, covering 14.5 river miles (23.3 kilometers) (Reference 2.4.4-7)

b. Guadalupe County, from the downstream limit of (a) to Dunlap Dam (5 miles or 8 kilometers below the city limit of New Braunfels, Texas), covering 17.1 miles (27.5 kilometers) (Reference 2.4.4-8)

c. From just upstream of Dunlap Dam to its confluence with Geronimo Creek, approximately 2500 feet (762.0 meters) downstream of FM-466, covering 22.8 river miles (36.7 kilometers) (Reference 2.4.4-9).

All the bridges, redundant cross sections (some associated with the bridges), diversions, and dams in the FIS HEC-RAS 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, Light Detection and Ranging data, local topographic maps, and field surveys) were developed to simulate the 100- and 500-year 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 miles or 4.0 kilometers upstream of the river mouth) were

based on the National Elevation Dataset (NED) model developed by the USGS (Reference 2.4.4-10). The length of this reach is about 239.3 river miles (385.1 kilometers). The cross sections were selected in such a way that they would be aligned in a generally perpendicular direction to the expected flow path of the dam breach flood wave.

The cross sections obtained from the USGS NED do not include definitive bathymetry of the river (under water/river bed 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 would yield 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 used in the HEC-RAS dam break model for the Guadalupe River are shown in Figure 2.4.4-4. Typical Guadalupe River model cross sections at four locations on the model river reach are shown in Figures 2.4.4-5 to 2.4.4-8.

For Coleto Creek, cross section data was obtained from two sources: (1) HEC-2 models developed for FEMA's FIS along the creek (Reference 2.4.4-11), and (2) NED data (Reference 2.4.4-10). 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 miles (20.9 kilometers), and were developed by Halff 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 (USBR) to incorporate changes in the channel cross sections and bridge details (Reference 2.4.4-12). 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.

Typical Coleto Creek model cross sections at two locations on the model river reach are shown in Figures 2.4.4-9 and 2.4.4-10.

#### 2.4.4.2.1.4 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 will experience in the river channel and

floodplain. Typically, different roughness coefficients are used to represent the channel and floodplain overbank 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 (Reference 2.4.4-7). The river segment in this study is primarily in non-urban floodplain conditions (unimproved channels and agricultural overbanks with scattered low-density residential areas). The Manning's roughness coefficient for the Guadalupe River varied from 0.035 to 0.040. Overbank 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 overbanks 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 Reference 2.4.4-7.

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 (Reference 2.4.4-8). For the overbank 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 Reference 2.4.4-8.

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 (Reference 2.4.4-9). Channel *n* values range from 0.02 to 0.10, and overbank *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. Overbank 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 overbanks 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 Reference 2.4.4-9.

For sections that were developed using the USGS NED data, a 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 overbank roughness values used in the calibrated flood studies summarized above.

#### 2.4.4.2.1.5 Predicted Water Levels at VCS Site from Upstream Dam Failure

For the HEC-RAS dam breach simulations, the downstream boundary condition is assumed to be at normal depth with a slope equal to 0.00016, as discussed in Subsection 2.4.3.

The HEC-RAS dam breach and unsteady flow routing models predicted that the peak water level in the Guadalupe River at the VCS site as a result of the failure of the Canyon and Coleto Creek dams

would be at elevations 47.1 feet (14.4 meters) NAVD 88 and 44.9 feet (13.7 meters) NAVD 88, respectively. These peak water levels do not include the wind wave effects. The peak discharges near the VCS site would be $1.33 \times 10^6$ cfs and $1.11 \times 10^6$ cfs, respectively. The flood waves would take about 23 hours and 1 hour, respectively, to reach the VCS site after the failure of Canyon and Coleto Creek Dams. The predicted dam break flood discharge and stage hydrographs near VCS for the two dam failure events are presented in Figures 2.4.4-11 and 2.4.4-12. The simulated maximum dam break water surface profiles from the Canyon and Coleto Creek Dams to the downstream boundary of the models are depicted in Figures 2.4.4-13 and 2.4.4-14, respectively.

#### 2.4.4.2.2 Cooling Basin Breach Analysis

The depth averaged two-dimensional (2-D) feature of the flow modeling software, Delft3D-FLOW (Reference 2.4.4-13), was used to evaluate the flooding potential because of the breaching of the cooling basin embankment. Delft3D-FLOW is a multidimensional hydrodynamic and transport numerical model, which simulates unsteady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary-fitted grid system. The model solves the Navier-Stokes equations for incompressible fluid using the shallow water and the Boussinesq assumptions. In addition, for 3-D simulations, the vertical turbulence eddy viscosity and turbulent diffusivity are computed by employing a turbulence closure model. The set of partial differential equations from the Navier-Stokes equations and the turbulence closure model are solved by using finite difference based numerical schemes.

Delft3D-FLOW 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 cooling basin, the model domain was delineated in such a way that the entire cooling basin is included, together with the areas surrounding the power block. The western, eastern, and northern boundaries of the model were selected considering that the maximum flood level would occur at the power block before the flood waves reach the boundaries and influence water levels at the power block area. The no-flow boundary condition was applied to the four external boundaries of the model domain.

The model domain covers an area of approximately 17,080 acres (6910 hectares): 8.7 miles (14,000 meters) in the east-west direction and about 9.1 miles (about 14,700 meters) in the north-south direction. Table 2.4.4-3 lists the coordinates of the four corners of the model domain. The numerical grid for the rectangular model was generated with the Delft3D-RGFGRID module: the horizontal grid size at the power block area for the VCS site is 32.8 feet by 32.8 feet (10 meters by 10 meters), the grid size for the areas away from the power block is 65.6 feet by 65.6 feet (20 meters by 20 meters), and the grid size for the transitional region is 32.8 feet by 65.6 feet (10 meters by 20

meters). Because the principal direction of the propagation of the flood waves is perpendicular to the cooling basin northern embankment, the model was also oriented in this direction. Figure 2.4.4-15 shows the numerical grid of the cooling basin embankment breach model. As part of a sensitivity test to demonstrate grid size independence, a coarser grid model was developed with 65.6 feet by 65.6 feet (20 meters by 20 meters) grid sizes, and the simulation results were compared with the finer grid model. The difference in the water level results of the two models was insignificant.

All embankments were modeled as “dry points” in which the flows perpendicular to the four faces of the grid cells, representing the embankments, are blocked. As part of the sensitivity test, the effect of blockage by the buildings west of the power block area was investigated. The model results show that the buildings do not have a significant impact on the estimated flood levels at the power block area. Therefore, the buildings west of the power block area were not included in the model, assuming any such buildings would be washed out by the breaching flood wave.

The embankment breach model evaluation is conducted for two simulation periods. The first model has a 30-minute simulation period and focuses on capturing the peak water level at the power block, which is expected to occur in the 30-minute period after the breach begins. The second evaluation extends the simulation to 96 hours and focuses on the water level and flow hydrographs during the recession period after the arrival of the flood peak at the power block.

#### 2.4.4.2.2.1 Assumptions in the Cooling Basin Breach Analysis

In the cooling basin breach analysis, the following assumptions were adopted:

1. The failure and removal of the breached section in the northern embankment would be instantaneous
2. All internal dikes within the cooling basin would also fail and be removed instantaneously, coinciding with the breaching of the northern embankment
3. The flow velocities in the cooling basin are zero before the instantaneous breach of the embankment

#### 2.4.4.2.2.2 Bathymetry Elevations of the Cooling Basin Breach Model

The model bathymetry, which is also the elevation of the bottom boundary, was established using: (1) USGS NED data (Reference 2.4.4-10), and (2) aerial survey data and a conceptual grading of the VCS site as shown in Figures 2.4.4-16 and 2.4.4-17. For the model area outside the coverage of the aerial survey and the grading plan, the USGS NED data was used and the interface between the two data sets is indicated in Figure 2.4.4-18. Bathymetric data was incorporated into the model with the Delft3D-QUICKIN module (Reference 2.4.4-13). Figures 2.4.4-19 and 2.4.4-20 show the model representation of the bathymetry for the entire model, and near the power block area, respectively.

Bathymetric data is referenced to NAVD 88, and, therefore, any ground elevation above NAVD 88 would be represented by a negative value in the Delft3D model.

The bed level of the cooling basin was set based on the proposed grading. The cooling basin bed level of the area between the northern embankment and about 11,600 feet (3537 meters) to the south was set to elevation 69 feet (21 meters) NAVD 88. For the remaining area of the cooling basin, the bed level was set according to the existing grades.

#### 2.4.4.2.2.3 Boundary Conditions of the Cooling Basin Breach Model

The rectangular model domain is bounded by four no-flow boundaries. The northern and western boundaries were positioned far enough downstream so that the maximum flood level at the VCS power block due to a cooling basin breach would occur before the flood wave front reaches the two boundaries and influences the water levels at the power block area.

#### 2.4.4.2.2.4 Initial Conditions of the Cooling Basin Breach Model

The breach model used to study the flood elevations considers that the cooling basin would be initially filled to elevation 93.9 feet (28.6 meters) NAVD 88, which corresponds to the one-half PMP condition (Subsection 2.4.8). The one-half PMP is selected based on the guideline provided in ANSI/ANS-2.8-1992 (Reference 2.4.4-3) for combined-events criteria for the evaluation of flooding due to failure of water impounding embankments. The one-half PMP is routed through the CB by conservatively assuming that the starting water level is the normal maximum operating water level of 91.5 feet (27.9 meters) NAVD 88. In addition, it is assumed that all the emergency spillway gates are closed or not in service. The maximum water level of 93.9 feet (28.6 meters) NAVD 88 obtained from the routing process is specified as the initial water level of the CB for the breach model.

Three initial downstream flood levels were considered as part of a sensitivity analysis: (1) elevation 82.0 feet (25.0 meters), (2) elevation 84.0 feet (25.6 meters), and (3) dry conditions. The sensitivity analysis concluded that the maximum flood level at the power block area is not sensitive to the initial downstream flood levels.

For the extended simulation, the dry bed initial condition was specified, as it enables the model to simulate more realistically the flow pattern in the recession period after the flood level at the power block reaches its peak. Based on the topographic setting at the VCS site, the breach flow leaving the cooling basin is expected to be diverted eventually towards lower terrain such as the adjacent Guadalupe River valley on the east and Kuy Creek on the west, as indicated in Figures 2.4.4-25 and 2.4.4-28. The other two initial conditions tested assume that the entire model domain (outside the cooling basin and the area bounded by the security wall) would be filled to 82 feet (25.0 meters) NAVD 88 and 84 feet (25.6 meters) NAVD 88. With the nominal natural grade in the power block

vicinity at about 80 feet (24.4 meters) NAVD 88, and lower farther to the east and west in the river valley and creeks, these initial water levels would potentially create a backwater effect and impede flow movement towards lower ground, especially during the recession period. The use of the dry bed initial condition for the extended simulation circumvents this modeling limitation.

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

#### 2.4.4.2.2.5 Selection of the Cooling Basin Breach Model Parameters

The surface roughness in the model was represented by Manning's *n* values. Based on the suggested values in Reference 2.4.4-14, Manning's *n* was specified as 0.030 uniformly in the two principal directions (east-west and north-south) throughout the model domain. The selected Manning's *n* corresponds to flood plain type terrains occupied by pastures with short grasses, comparable to the general characteristics of the existing conditions at the power block area and vicinity. After construction, the ground between the cooling basin and the power block would consist of paved areas, such as parking lots (see Figure 1.2-2), and gravel/grass covered areas with lower surface roughness. For example, the suggested Manning's *n* values range from 0.013 to 0.016 for asphalt-lined channels and 0.022 to 0.033 for engineered channels with gravel and grass covers (Reference 2.4.4-14). To address the differences in the surface roughness and potential impact on flow resistance, a sensitivity analysis was performed to evaluate the effect of the Manning's *n* on the flood level at the power block area. Manning's *n* values ranging from 0.01 to 0.2 were tested, and the results show that the flood water levels at the power block area are not sensitive to the Manning's *n* values.

The simulations were run for a 30-minute period at a model time step of 0.01 minutes (0.6 seconds), which was selected based on a verification effort to demonstrate that the time step would be independent of the model results. For the extended period simulation of 96 hours, a time step of 0.04 minutes (2.4 seconds) was adopted. The maximum water levels obtained from the two time steps were similar with no noticeable difference.

#### 2.4.4.2.2.6 Flood Levels from the Cooling Basin Breaches

As shown in Figures 2.4.4-16 and 2.4.4-17, the distance between the northern embankment of the cooling basin and the security wall is over 500 feet (150 meters). The area in between is built up (with fill materials) to elevations in the general range of 86 feet (26.2 meters) to 102 feet (31.1 meters) NAVD 88, from the natural grade of about 80 feet (24.4 meters) NAVD 88. A local drainage ditch south of the security wall has invert elevations slightly lower than 86 feet (26.2 meters) NAVD 88. Specifically:

- Along the mid-section, the grade is filled to 102 feet (31.1 meters) NAVD 88 immediately north of the embankment, decreasing to about 92 feet (28.0 meters) NAVD 88 at 400 feet (122 meters) away at the northern edge of the parking lot, before the drainage ditch and the security wall. The grade elevation gradually decreases towards the east and west from the mid-section.

- Along the western end of the security wall, the grade elevation is approximately 94 feet (28.6 meters) NAVD 88 immediately north of the embankment, dropping to 93 feet (28.3 meters) NAVD 88 at 400 feet (122 meters) away to the north.

- Similarly, along the eastern end of the security wall, the grade is filled to 93 feet (28.3 meters) NAVD 88 immediately north of the embankment, and to 92 feet (28.0 meters) NAVD 88 at 400 feet (122 meters) away to the north.

As described above, most of this built up area is higher than or has similar elevation to the initial cooling basin level of 93.9 feet (28.6 meters) NAVD 88 for the postulated embankment breaches. It is therefore highly unlikely that the cooling basin will breach through the large amount of fill materials placed between the cooling basin and the security wall. Further towards the east and west, the grade elevations continue to decrease gradually, and the probability that a breach might occur increases slightly. Conservatively, the breach analysis postulates that the embankment would fail where the grade elevation immediately to the north of the embankment is below elevation 96.0 feet (29.3 meters) NAVD 88 on the west and 94.0 feet (28.7 meters) NAVD 88 on the east as shown in Figures 2.4.4-16 and 2.4.4-17, respectively. Therefore, two breaching scenarios were considered: (1) cooling basin embankment breaching west of the power block (western breach) area, and (2) cooling basin embankment breaching east of the power block (eastern breach) area.

Multiple embankment breach widths were investigated with the Delft3D model by removing the “dry points” that represent the embankments. The breach widths simulated for the western breach scenario vary from 689 feet (210 meters) to 2034 feet (620 meters). The breach widths simulated for the eastern breach scenario vary from 2329 feet (710 meters) to 3970 feet (1210 meters). The resulting maximum flood levels for the various simulated breach widths are presented in Tables 2.4.4-4 and 2.4.4-5. For the western breach, the maximum flood level, at elevation 90.6 feet (27.6 meters) NAVD 88, would occur at the southwestern corner of the security wall. For the eastern breach, the maximum flood level, at elevation 89.8 feet (27.4 meters) NAVD 88, would occur near the midsection of the security wall on the south side. The simulated maximum water levels as the result of embankment breach are lower than the minimum site grade of the power block.

Figures 2.4.4-21 and 2.4.4-22 detail the time history of the simulated flood level at the southwestern corner of the security wall and near the midsection of the security wall on the south side, respectively (model simulations begin at an arbitrarily selected date of 26-April-2008). As indicated in the figures,

the flood wave arrives at the southwestern corner of the security wall and the southern entrance to the power block area in about 1 and 14 minutes after the embankment breaches, respectively. Figure 2.4.4-21a provides the predicted water level hydrograph of the extended simulation run, for the first 150 minutes after the postulated breach, at the southwest corner of the security wall where the maximum power block flood level occurs. The maximum still water level occurs within the first 30 minutes of embankment breach. Figure 2.4.4-21b details the water level hydrograph for the entire extended simulation period of 96 hours. As shown in that figure, the water level drops rapidly in the 12 hours past the peak and then more gradually to about 86.3 feet (26.3 meters) NAVD 88 at the end of 96 hours. The reason for the water level not returning to the initial level (i.e., the grade elevation of about 83 feet [25.3 meters] NAVD 88) is that this location of the maximum water level is situated in a drainage ditch, which is surrounded by higher ground of 86 feet (26.2 meters) NAVD 88 and above. Flood water is detained inside this local depression and presents no flood risk to the power block. Figure 2.4.4-21c details the time history of flow leaving the cooling basin. As the figure indicates, the flow leaving the cooling basin is negligible near the end of 96 hours of the postulated embankment breach.

Figures 2.4.4-23, 2.4.4-24, and 2.4.4-25 detail the water level and depth of flow contours, as well as velocity vectors, at the vicinity of the western breaching location, respectively, after 18 minutes of the cooling basin breach.

Figures 2.4.4-26, 2.4.4-27, and 2.4.4-28 detail the water level and depth of flow contours, as well as the velocity vectors, at the vicinity of the eastern breaching location, respectively, after 17 minutes of the cooling basin breach.

Although the highly conservative cooling basin breach flooding levels have a short time scale and would not sustain for a period long enough for any considerable wind-wave action, coincidental wind wave runup was considered for additional conservatism, and the results are provided in Subsection 2.4.4.3.2.

As Figures 2.4.4-23, 2.4.4-24, 2.4.4-26, and 2.4.4-27 illustrate, the maximum flood levels, as a result of the breaching of the cooling basin northern embankment, are lower than the minimum site grade at the power block.

### 2.4.4.3 Water Level at the VCS Site

Analyses of the dam failures within the Guadalupe River basin and the failure of the cooling basin northern embankment showed that the critical flood level at the power block is controlled by the cooling basin embankment failure. The power block site grade would be above the maximum flood level from the potential dam failures as explained below.

#### 2.4.4.3.1 Water Level at the VCS Site from the Failures of Upstream Dams

In accordance with the guidelines in ANSI/ANS-2.8-1992 (Reference 2.4.4-3), 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 storm surge (PMSS) conditions due to the probable maximum hurricane (PMH) is adopted here. As discussed in Subsection 2.4.5, the estimated wind setup and wave run-up is 21.3 feet (6.5 meters). The maximum flood level in the Guadalupe River near the VCS site as a result of the probable worst case upstream dam failure scenario coincidental with worst-case wind conditions (PMH) is estimated to be at elevation 68.4 feet (20.8 meters) NAVD 88. This flood level is lower than the minimum power block finished site grade of 95 feet (29.0 meters) NAVD 88.

#### 2.4.4.3.2 Water Level at the VCS Site from Breaching of the Cooling Basin

The predicted maximum still water level at the power block area for the postulated breaching of the cooling basin northern embankment is at elevation 90.6 feet (27.6 meters) NAVD 88. In accordance with the guidelines in ANSI/ANS-2.8-1992 (Reference 2.4.4-3), the flood level at the plant site considers the effects of wave runup from the coincidental occurrence of a 2-year design wind event. As shown in Figure 1 of ANSI/ANS-2.8-1992 (Reference 2.4.4-3), the 2-year fastest mile wind speed at 30 feet (9.2 meters) above ground at the VCS site is 50 mph (80.5 km/hr). This value is adjusted for duration, wind speed above water, and fetch length, as applicable, to estimate wave runup on the swale slope surrounding the power block area. Methodologies described in the *U.S. Army Corps of Engineers Coastal Engineering Manual* (Reference 2.4.4-18) were used to determine the wave height, wave period, and thus wave runup elevation. In the wind wave analysis, a fetch length of 18,682 feet (5694.3 meters) (the maximum distance between the location of the maximum water level and the southern embankment of the cooling basin), cooling basin bottom at elevation 69 feet (21.0 meters) NAVD 88, maximum swale slope of 3 percent, and top of security wall at elevation 90.0 feet NAVD 88 (which would limit the height of waves approaching the power block area) were used. Based on these parameters and values, the following results were obtained: the minimum wind duration is approximately 70 minutes for the given fetch, the spectral peak wave period is approximately 2.9 seconds, and maximum wave runup is approximately 0.3 feet (0.09 meters). Therefore, the maximum water level adjacent to the power block area of VCS is elevation 90.9 feet (27.7 meters) NAVD 88. After rounding the predicted maximum water level, a value of elevation 91.0 feet (27.7 meters) NAVD 88 is adopted. This maximum flood level is below the minimum power block finished site grade at elevation 95 feet (29.0 meters) NAVD 88.

#### 2.4.4.3.3 Sedimentation and Erosion

An upstream dam failure event would not flood the VCS site, as is evidenced by the model prediction, which shows that the maximum water level of elevation 68.4 feet (20.8 meters) NAVD 88 for this event is much lower than the natural grade of about 80 feet (24.4 meters) at the VCS power block. As such, the area that would be inundated by the flood flow is at a considerable distance away from the VCS power block. Therefore, no sedimentation or erosion affecting the power block of VCS is expected.

The power block area is not subject to flood inundation during the breaching of the cooling basin, and the predicted maximum water level is 4 feet (1.2 meters) below the minimum finished site grade of the power block. Therefore, the impact of sedimentation and erosion on the safety functions of VCS is not a concern.

### 2.4.4.4 References

2.4.4-1 Texas Administrative Code – Title 30, Part 1, §299.1 and §299.14; effective May 13, 1986.

2.4.4-2 Texas Water Development Board, *Water for Texas – 2007*, Vol. I, and II, January 2007.

2.4.4-3 American Nuclear Society, *Determining Design Basis Flooding at Power Reactor Sites,* ANSI/ANS-2.8-1992, July 1992. (Historical technical reference)

2.4.4-4 Texas Water Development Board, *Engineering Data on Dams and Reservoirs in Texas*, Part III, Report 126, February 1971.

2.4.4-5 U.S. Army Corps of Engineers (USACE), Hydrologic Engineering Center, HEC-RAS, River Analysis System, version 3.1.3, May 2005.

2.4.4-6 Federal Energy Regulatory Commission, “Appendix II-A: Dambreak Studies,” of “Chapter II: Selecting and Accommodating Inflow Design Floods for Dams,” *Engineering Guidelines for the Evaluation of Hydropower Projects*, October 1993.

2.4.4-7 Halff Associates, Inc., *Technical Support Data Notebook, Comal County,* Texas Phase I, Section II, Binder No. 3, and accompanying HEC-RAS model submitted to FEMA Region VI, October 11, 2004.

2.4.4-8 CH2M HILL, Inc., *FEMA Hydrologic and Hydraulic Analyses, Milestone No. 3 Submittal,* Hydraulic Modeling and Flood Plain Mapping, and accompanying HEC-RAS model submitted to City of New Braunfels, December 2002.

2.4.4-9 Halff Associates, Inc., *Technical Support Data Notebook, Guadalupe County, Texas Phase I,* Section II, Binder No. 2, and accompanying HEC-RAS model submitted to FEMA Region VI, revised on February 23, 2006.

2.4.4-10 U.S. Geological Survey (USGS), USA 30m National Elevation Dataset (NED) in ESRI RASTER GRID format, purchased on a portable hard-drive from Digital Data Services, Inc. on May 22, 2007.

2.4.4-11 Federal Emergency Management Agency (FEMA), *Flood Insurance Study, City of Victoria, Victoria County, Texas (Unincorporated Areas)*, Washington, D.C., May 17, 1990; and supporting HEC-2 model for Coleto Creek developed by Halff Associates, Inc.

2.4.4-12 Federal Emergency Management Agency, *Flood Insurance Study, City of Victoria, Victoria County, Texas (Unincorporated Areas)*, Washington, D.C., revised November 20, 1998; and supporting HEC-2 model for Coleto Creek developed by USGS, Water Resources Division.

2.4.4-13 WL|Delft, *2005: Delft3D-FLOW, Simulation of Multi-Dimensional Hydrodynamic Flows and Transport Phenomena, Including Sediments: User Manual*, *Delft,* Netherlands, 2005.

2.4.4-14 Chow, Ven Te, *Open Channel Hydraulics*, McGraw-Hill Book Company, New York, New York, 1959.

2.4.4-15 NOAA National Geodetic Survey, *Orthometric Height Conversion*, Available at http://geodesy.noaa.gov/cgi-bin/VERTCON/vert_con.prl, accessed on February 27, 2008 and April 28, 2008.

2.4.4-16 U.S. Army Corps of Engineers (USACE), “Fort Worth District,” *Dam Assurance Study on Canyon Lake, Guadalupe Basin,* with input and output data files for Watershed Run-off Computer Model (WRCM), June 2005.

2.4.4-17 URS Corporation, *Bi-Annual Dam Inspection of Coleto Creek Dam*, submitted to Guadalupe-Blanco River Authority, August 21, 2003.

2.4.4-18 U.S. Army Corps of Engineers, Coastal Hydraulics Laboratory, EM1110-3-1100, *Coastal Engineering Manual*, 2008.

**Table 2.4.4-1 (Sheet 1 of 2)**
**Summary of Dams in the Guadalupe River Basin with Storage Capacity of 3000 AF or More**

| No. | Dam Name | County | Height of Dam (feet) | Length of Dam (feet) | Top of Dam Elevation (feet, MSL[a]) | Maximum Capacity (acre-feet) | Dam Type | Date of Completion |
|---|---|---|---|---|---|---|---|---|
| 1 | Canyon Dam | Comal | 219[b] | 6830 | 974.0 | 1,129,300[c] | Earth | 1964 |
| 2 | Comal River WS SCS Site 4 Dam | Comal | 73 | 2000 | 806.3 | 5293 | Earth | 1965 |
| 3 | York Creek WS SCS Site 1 Dam | Comal | 81 | 1157 | 742.8 | 4570 | Earth | 1967 |
| 4 | Comal River WS SCS Site 3 Dam | Comal | 58 | 1850 | 783.3 | 6911 | Earth | 1974 |
| 5 | Plum Creek WS SCS Site 5 Dam | Hays | 38 | 2510 | 668.0 | 3368 | Earth | 1963 |
| 6 | Plum Creek WS SCS Site 6 Dam | Hays | 36 | 3340 | 643.1 | 5663 | Earth | 1967 |
| 7 | York Creek WS SCS Site 5 Dam | Hays | 41 | 1897 | 589.0 | 3426 | Earth | 1963 |
| 8 | Lake Meadow Dam | Guadalupe | 27 | 2525 | 475.6 | 3100 | Earth | 1930 |
| 9 | Lake Placid Dam[d] | Guadalupe | 25 | 2057 | — | 5400 | Gravity, earth | 1964 |
| 10 | Lake Mcqueeney Dam | Guadalupe | 40 | 1555 | 540.0 | 5050 | Earth | 1928 |
| 11 | Lake Dunlap Dam | Guadalupe | 41 | 1626 | 589.4 | 5900 | Earth | 1928 |
| 12 | York Creek WS SCS Site 13 Dam | Guadalupe | 33 | 2782 | 595.3 | 5045 | Earth | 1964 |
| 13 | Lake Gonzales Dam | Gonzales | 42 | 2170 | 346.5 | 23,520 | Earth | 1931 |
| 14 | Lake Wood Dam | Gonzales | 42 | 6450 | 304.0 | 8120 | Earth | 1931 |
| 15 | Lower Plum Creek WS SCS Site 34 Dam | Caldwell | 41 | 3106 | 573.6 | 4741 | Earth | 1965 |
| 16 | Lower Plum Creek WS SCS Site 28 Dam | Caldwell | 34 | 4300 | 479.5 | 5404 | Earth | 1963 |
| 17 | Plum Creek WS SCS Site 14 Dam | Caldwell | 46 | 3640 | 542.3 | 8715 | Earth | 1967 |
| 18 | Plum Creek WS SCS Site 17 Dam[d] | Caldwell | 35 | 1860 | — | 5312 | Earth | 1969 |

**Table 2.4.4-1 (Sheet 2 of 2)**
**Summary of Dams in the Guadalupe River Basin with Storage Capacity of 3000 AF or More**

| No. | Dam Name | County | Height of Dam (feet) | Length of Dam (feet) | Top of Dam Elevation (feet, MSL[a]) | Maximum Capacity (acre-feet) | Dam Type | Date of Completion |
|---|---|---|---|---|---|---|---|---|
| 19 | Plum Creek WS SCS Site 21 Dam | Caldwell | 41 | 3400 | 522.3 | 5318 | Earth | 1962 |
| 20 | Comal River WS SCS Site 1 Dam | Comal | 70 | 2530 | 919.3 | 6763 | Earth | 1978 |
| 21 | Plum Creek WS SCS Site 16 Dam | Hays | 41 | 2800 | 559.9 | 3642 | Earth | 1975 |
| 22 | Lower Plum Creek WS SCS Site 27 Dam[d] | Caldwell | 28 | 3830 | — | 3170 | Earth | 1974 |
| 23 | Coleto Creek Dam | Victoria | 65 | 21,000 | 120.0 | 169,000[e] | Earth | 1980 |
| 24 | Comal River WS SCS Site 2 Dam | Comal | 75 | 3100 | 866.8 | 19,024 | Rockfill | 1981 |
| 25 | Upper San Marcos River WS SCS Site 1[d] | Hays | 80 | 2905 | — | 18,399 | Earth | 1983 |
| 26 | Upper San Marcos River WS SCS Site 2 | Hays | 51 | 1465 | 726.7 | 3034 | Earth | 1985 |
| 27 | Upper San Marcos River WS SCS Site 4 | Hays | 100 | 1365 | 889.8 | 5972 | Earth | 1985 |
| 28 | Upper San Marcos River WS NRCS Site 5 Dam | Hays | 71 | 2950 | 667.2 | 7329 | Earth | 1989 |
| 29 | Upper San Marcos River WS SCS Site 3[d] | Hays | 60 | 1630 | — | 4323 | Earth | 1991 |

(a) MSL datum is equivalent to NGVD 29 and is assumed to be the same as NAVD 88 as the result of a very small difference.
(b) Actual dam height is 224 feet (Reference 2.4.4-4).
(c) Top-of-dam storage of about 1.2 MAF was used in the dam break analysis based on the elevation-storage curve given in Reference 2.4.4-4.
(d) Top-of-dam elevation is not available in the TCEQ dam database.
(e) Top-of-dam storage of 149,800 AF, as discussed in Subsection 2.4.3 was used in the dam break analysis.

**Table 2.4.4-2**
**Breach Parameters for Canyon and Coleto Creek Dams**

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

Note: MSL datum is equivalent to NGVD 29 and is assumed to be the same as NAVD 88 due to the very small difference.

**Table 2.4.4-3**
**Coordinates of Cooling Basin Breach Model Domain Corner Points**

| Model Corners | Easting (feet/meters) | Northing (feet/meters) |
|---|---|---|
| Southwest | 2612458.5/796277.4 | 13380265.3/4078304.9 |
| Southeast | 2641982.9/805276.4 | 13415451.1/4089029.5 |
| Northwest | 2575513.4/785016.5 | 13411265.9/4087753.8 |
| Northeast | 2605037.8/794015.5 | 13446451.7/4098478.5 |

Based on NAD 83, Texas State Plane South Central

**Table 2.4.4-4**
**Variation of Maximum Flood Level at the VCS, with Different Cooling Basin Western Breach Widths**

| Breach Width | | Maximum Water Level (NAVD 88) | |
|---|---|---|---|
| **Meters** | **Feet** | **Meters** | **Feet** |
| 210 | 689 | 27.42 | 90.0 |
| 310 | 1017 | 27.61 | 90.6 |
| 360 | 1181 | 27.62 | 90.6 |
| 410 | 1345 | 27.62 | 90.6 |
| 620 | 2034 | 27.60 | 90.6 |

**Table 2.4.4-5**
**Variation of Maximum Flood Level at the VCS, with Different Cooling Basin Eastern Breach Widths**

| Breach Width | | Maximum Water Level (NAVD 88) | |
|---|---|---|---|
| **Meters** | **Feet** | **Meters** | **Feet** |
| 710 | 2329 | 27.06 | 88.8 |
| 810 | 2657 | 27.25 | 89.4 |
| 910 | 2986 | 27.34 | 89.7 |
| 1010 | 3314 | 27.36 | 89.8 |
| 1210 | 3970 | 27.21 | 89.3 |

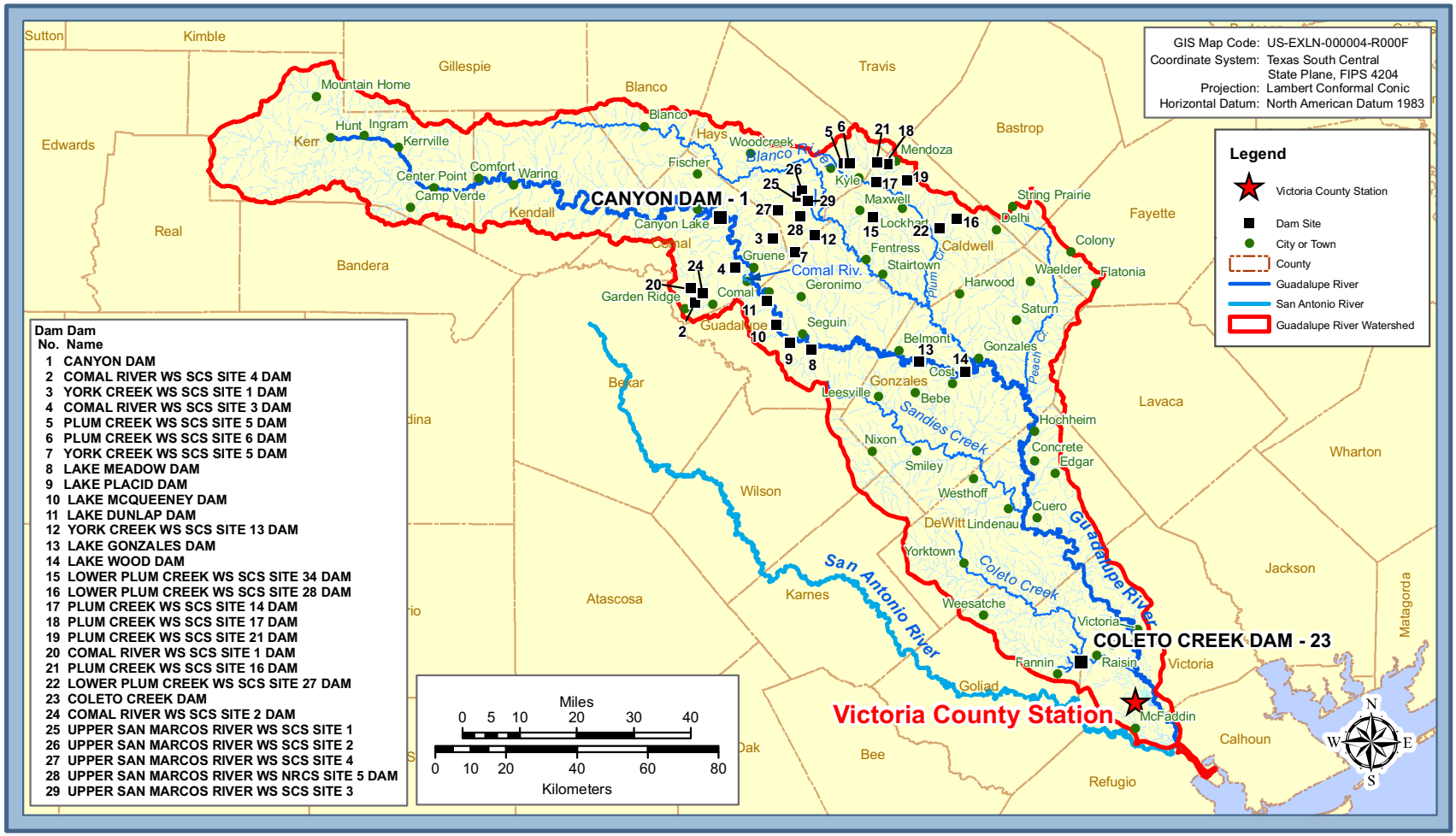
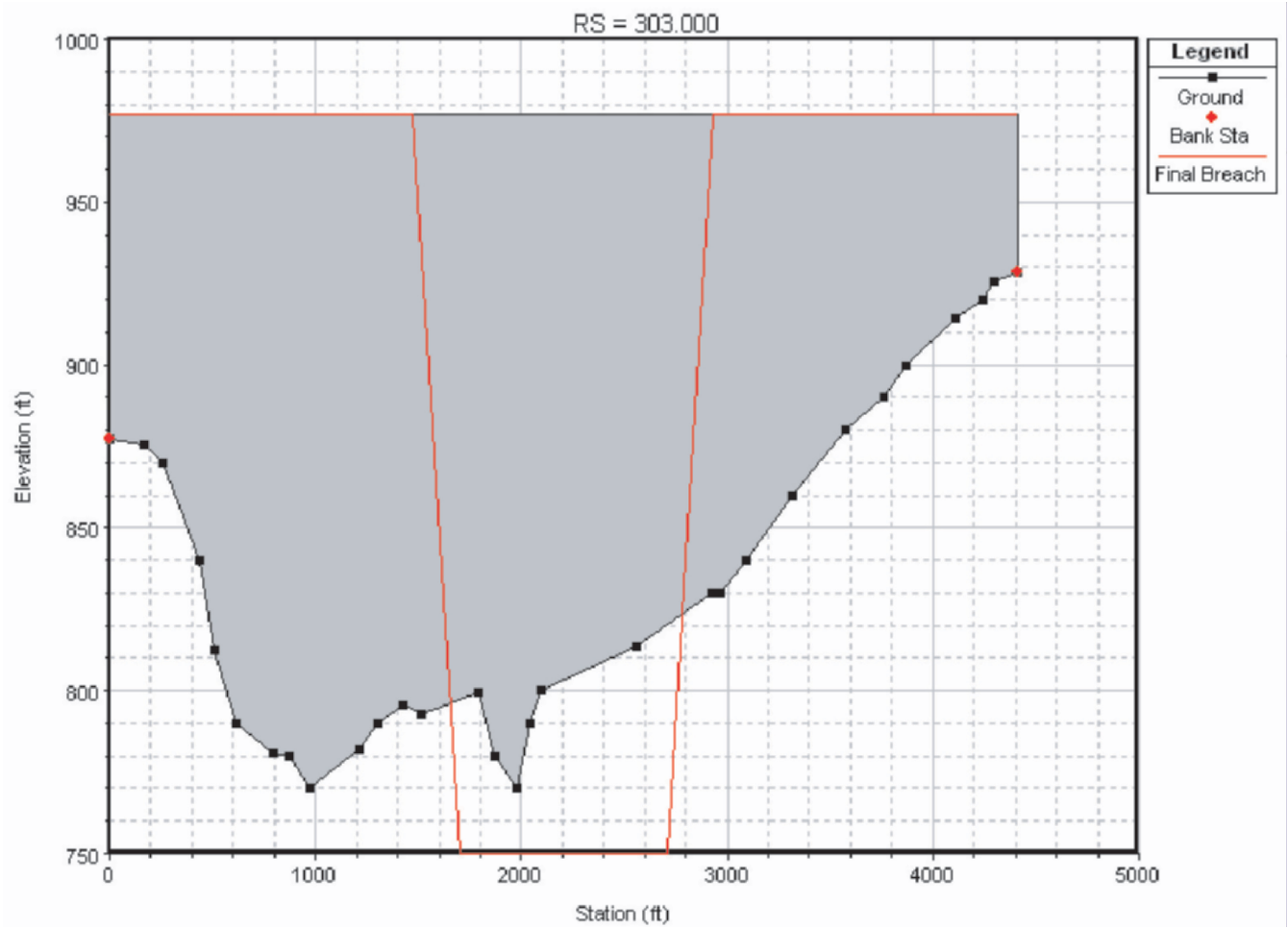
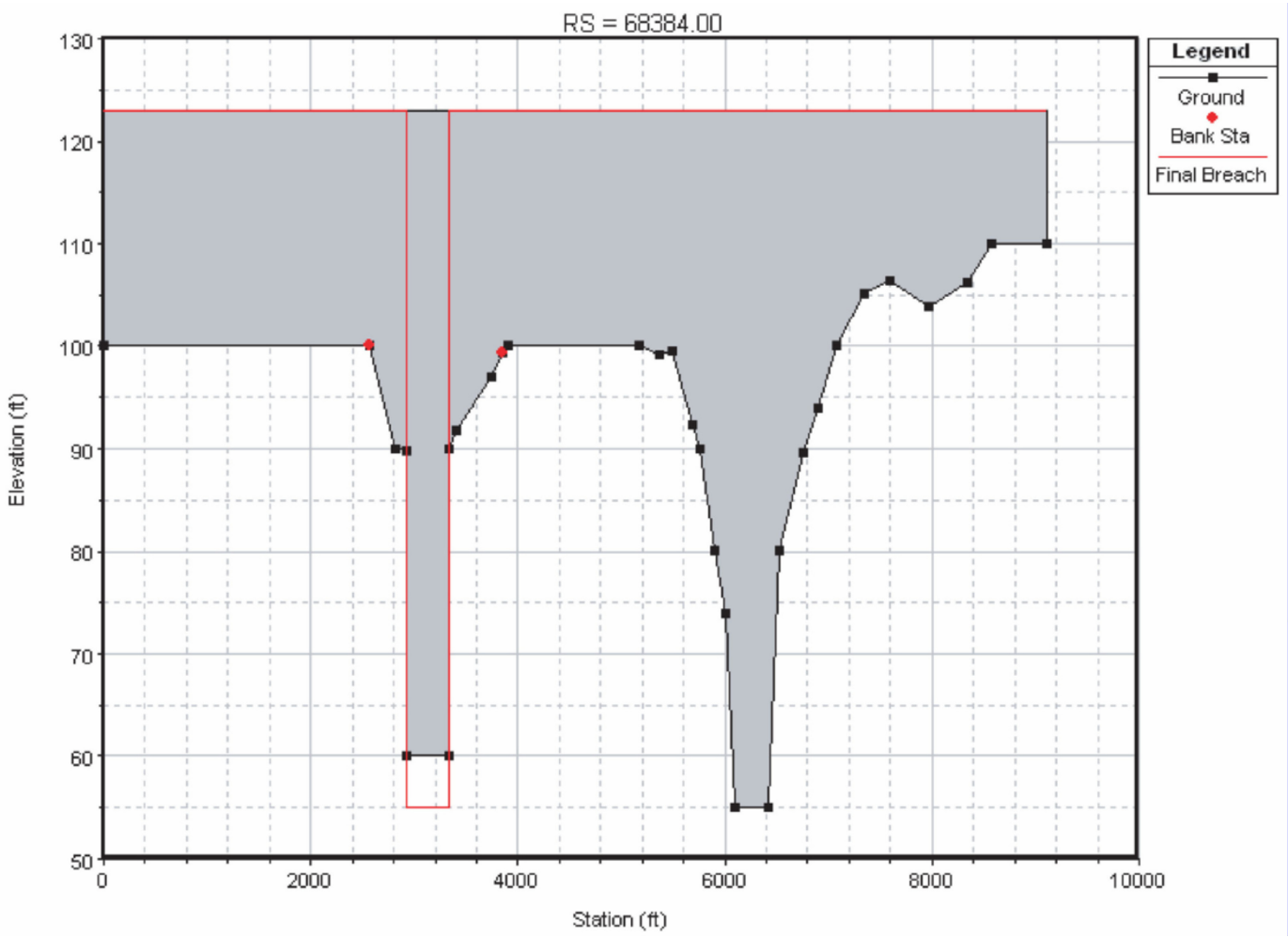



**Figure 2.4.4-1 Location of Dams in the Guadalupe River Basin with Storage Volume of 3000 AF or More**




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

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




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

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

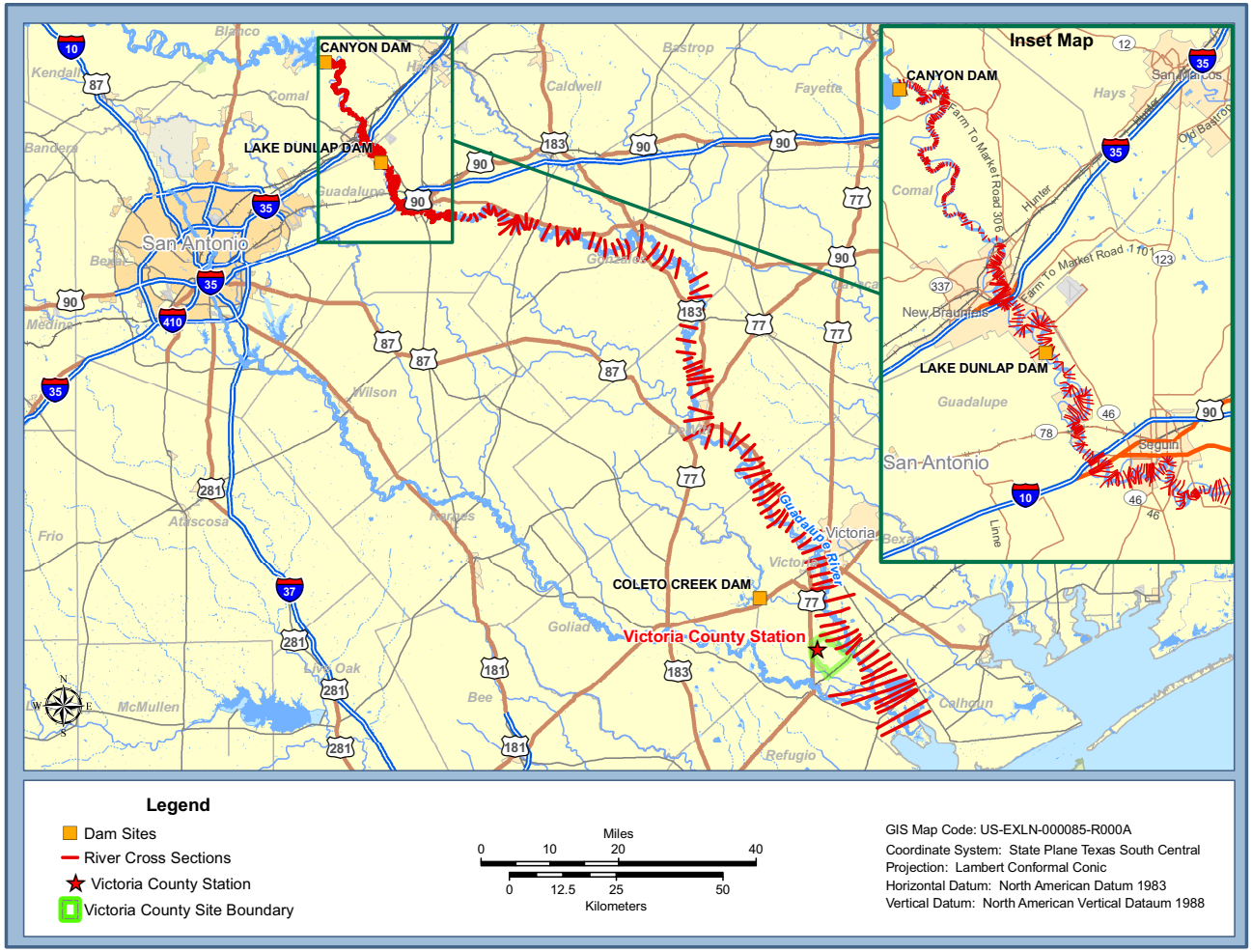
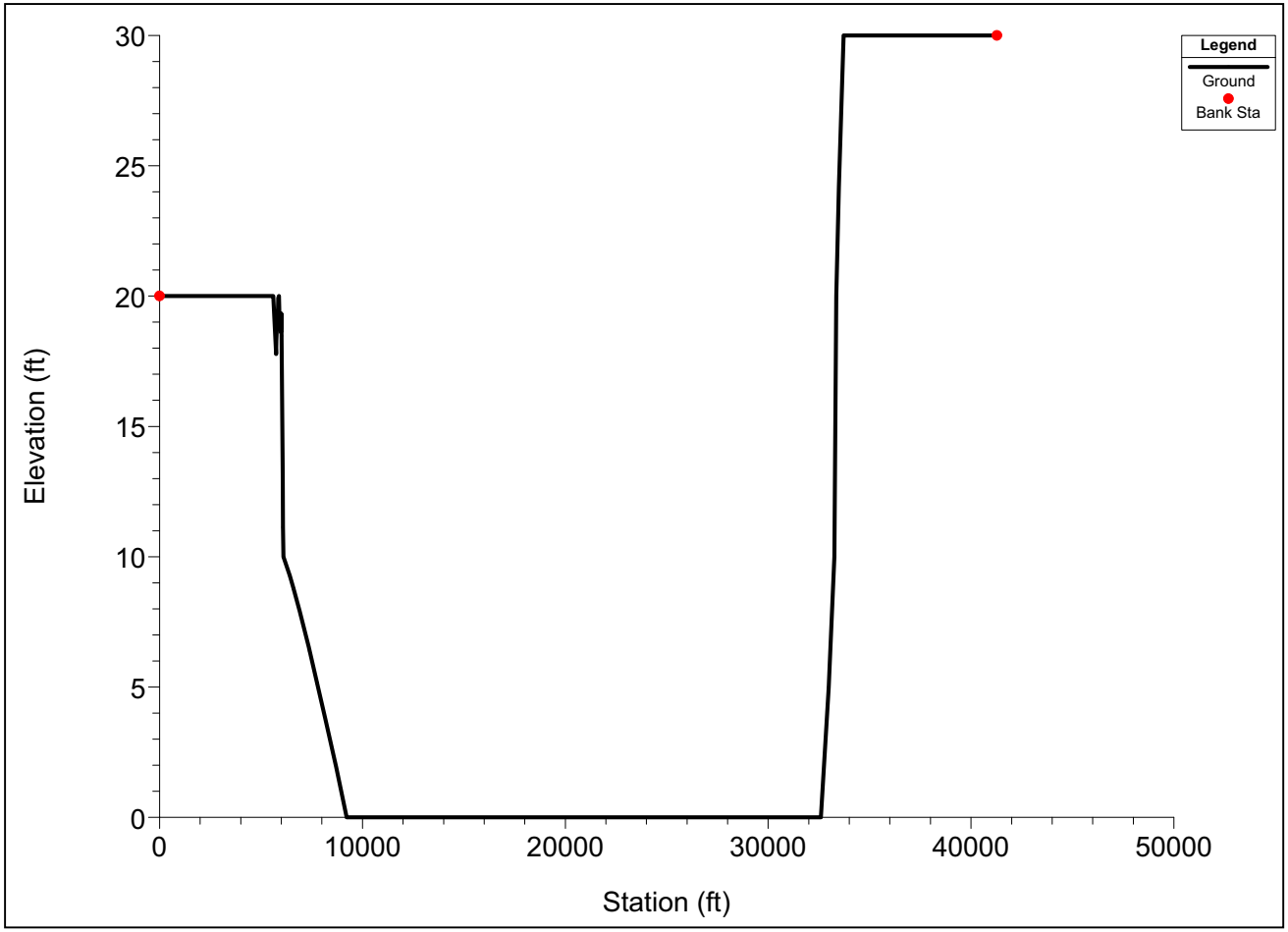
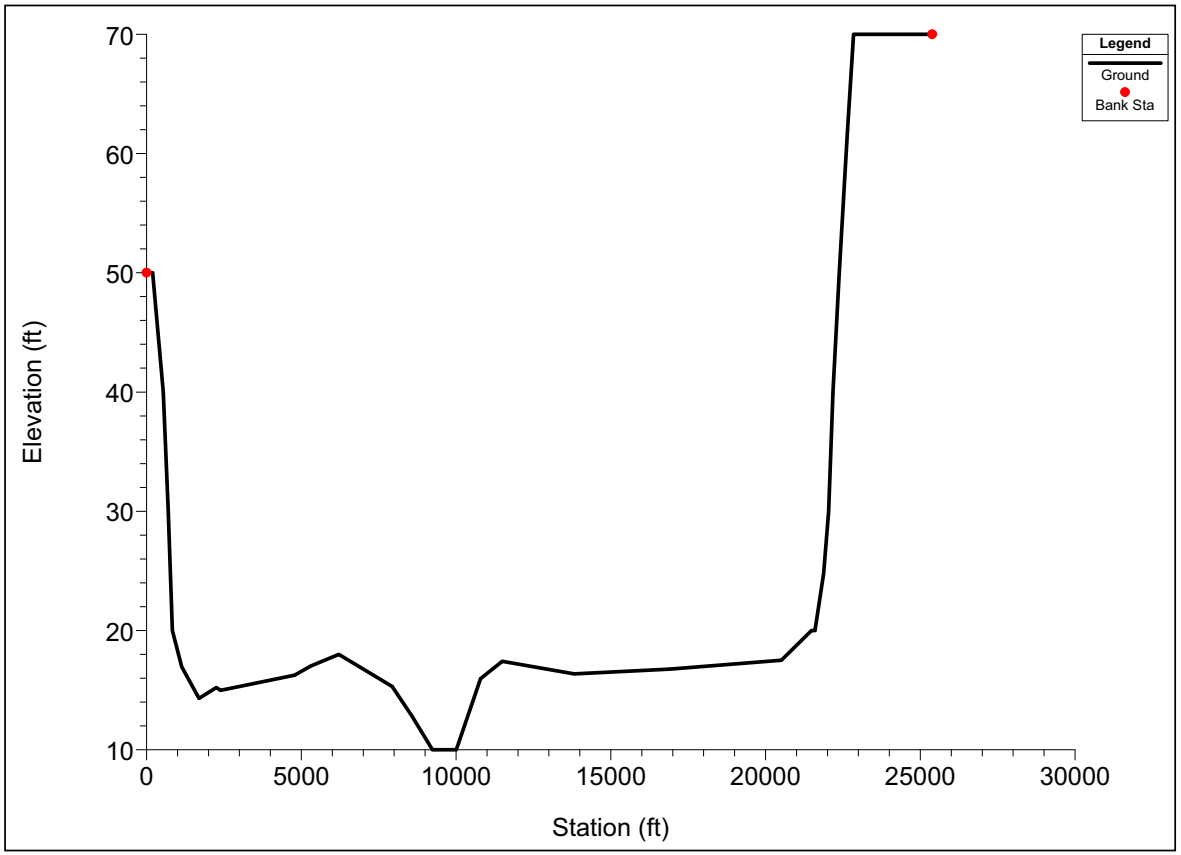



**Figure 2.4.4-4 Plan View of Cross Section Locations on the Guadalupe River**



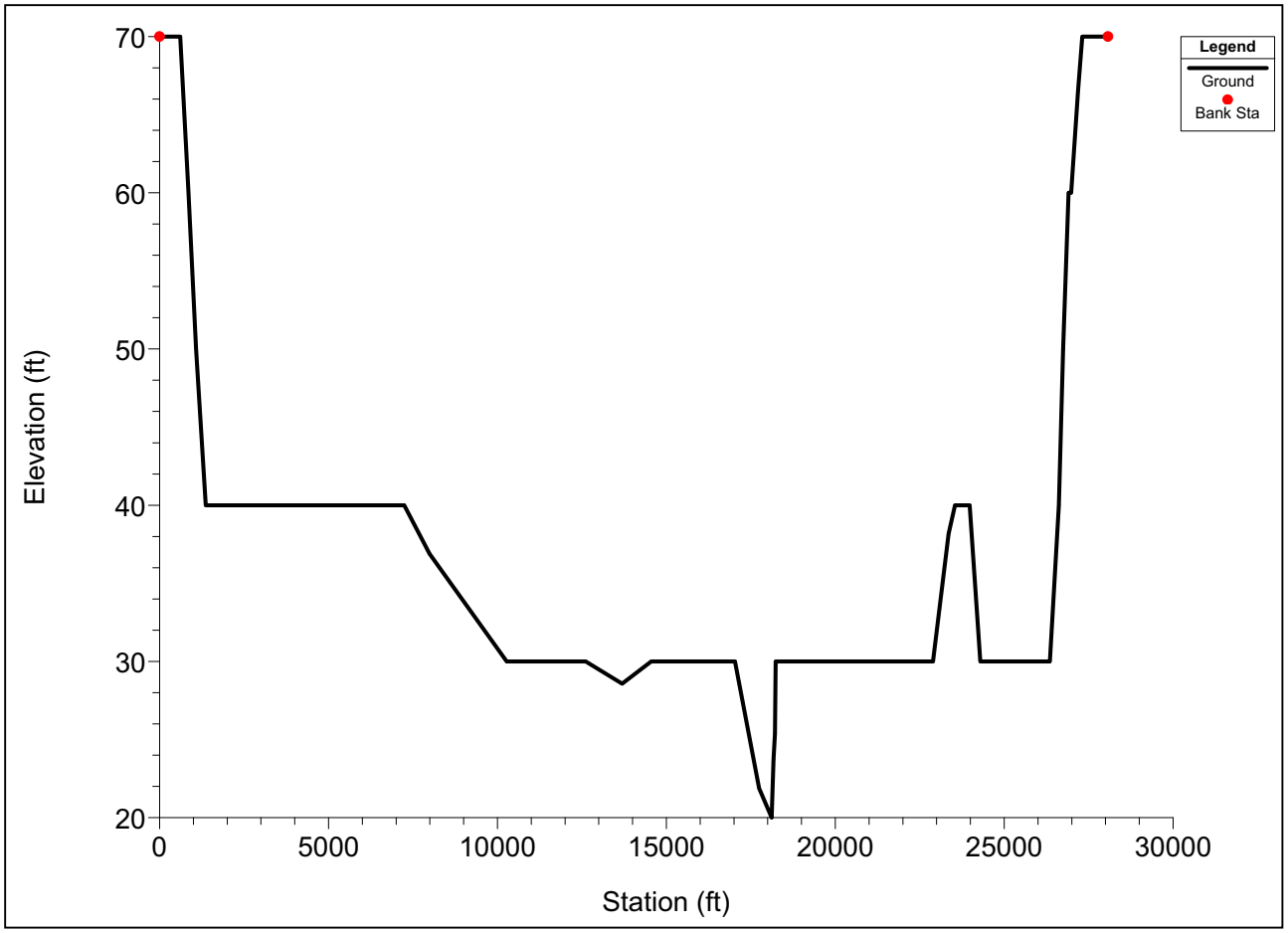

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-5 Guadalupe River Model Cross Section at Downstream Boundary**



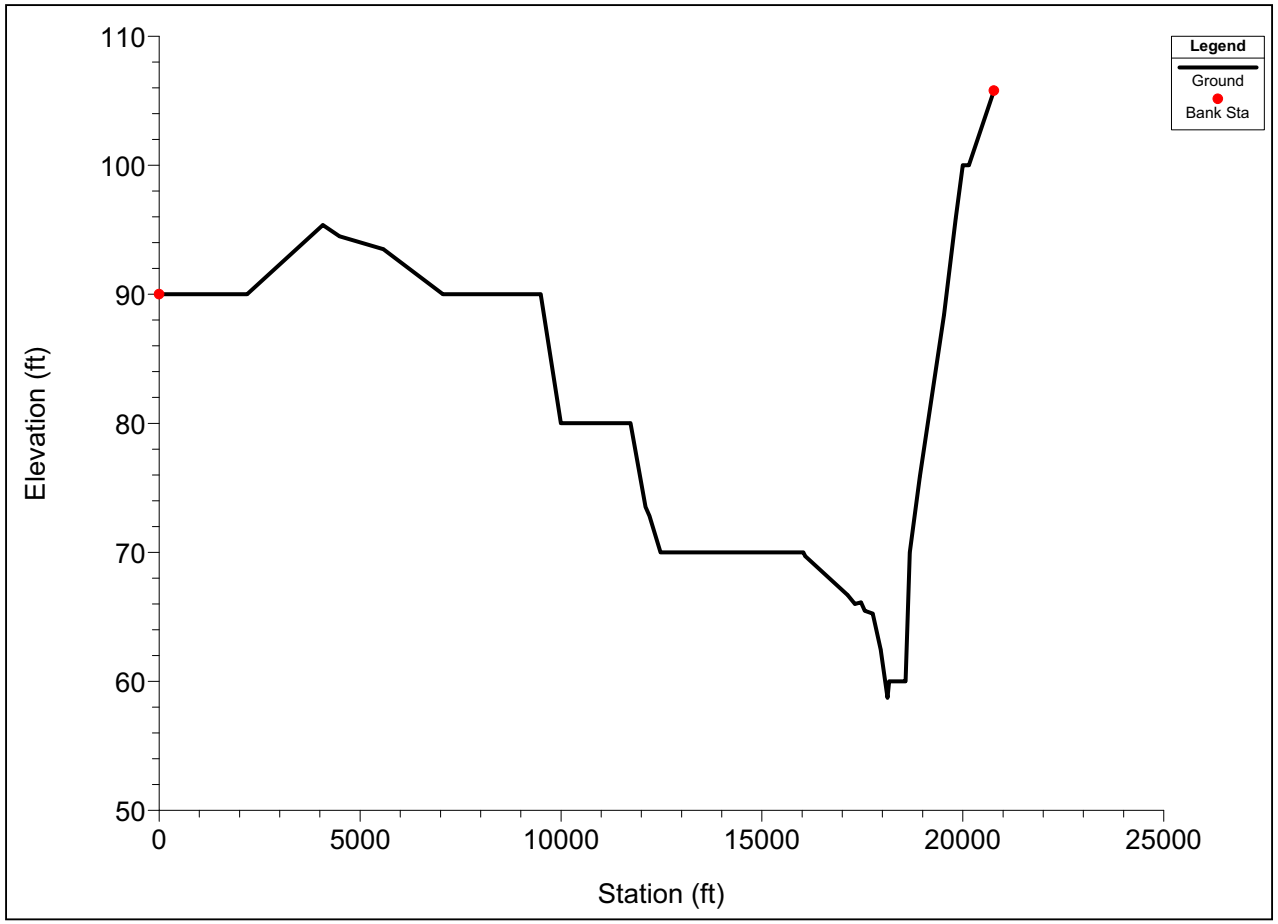

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-6 Guadalupe River Model Cross Section near VCS at about 22.4 River Miles Upstream of the Downstream Boundary**



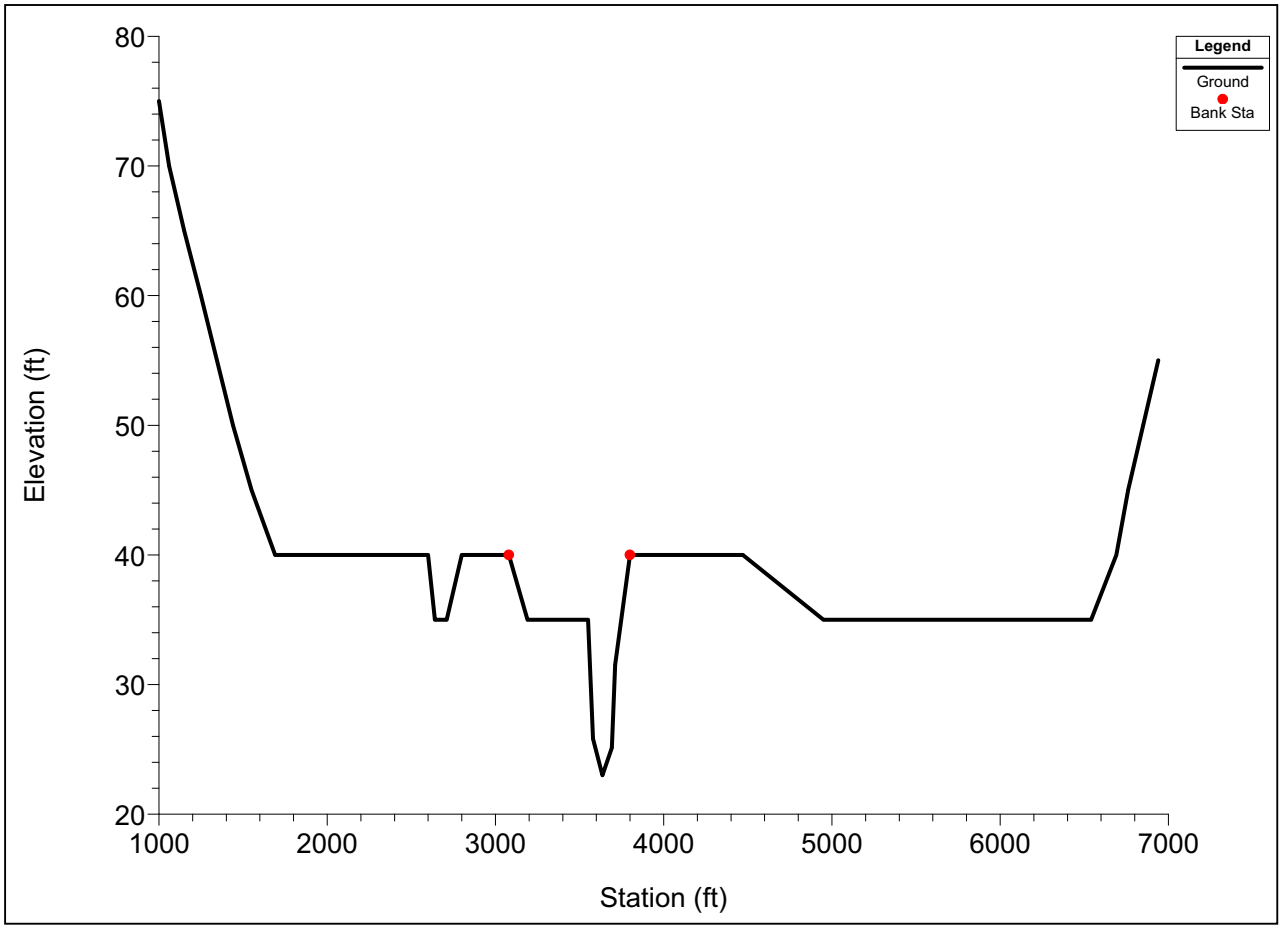

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-7 Guadalupe River Model Cross Section at about 34.4 River Miles from the Downstream Boundary**



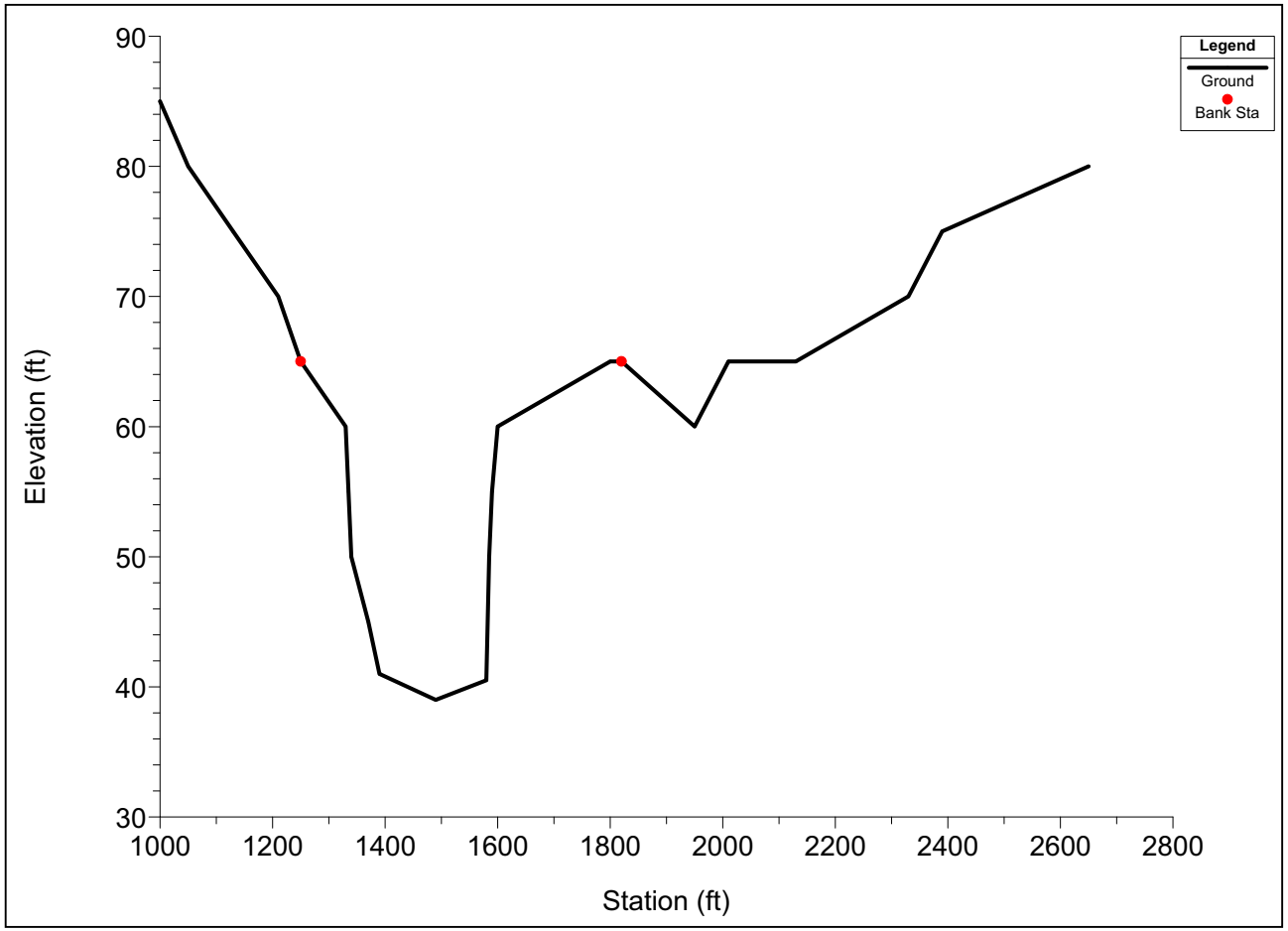

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-8 Guadalupe River Model Cross Section at about 56.7 River Miles from the Downstream Boundary**



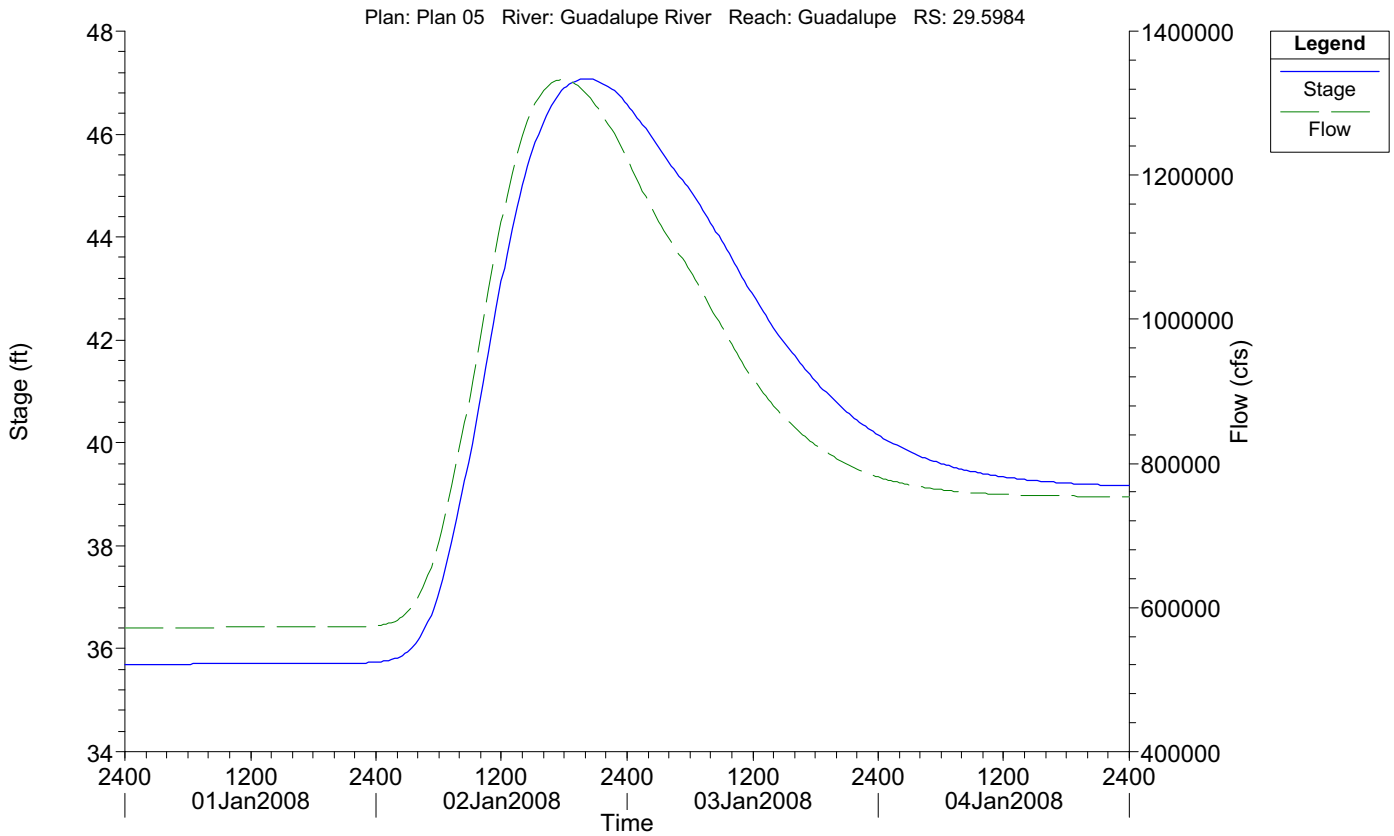

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-9 Coleto Creek Model Cross Section at about 1.6 River Miles Upstream of the Confluence with the Guadalupe River**



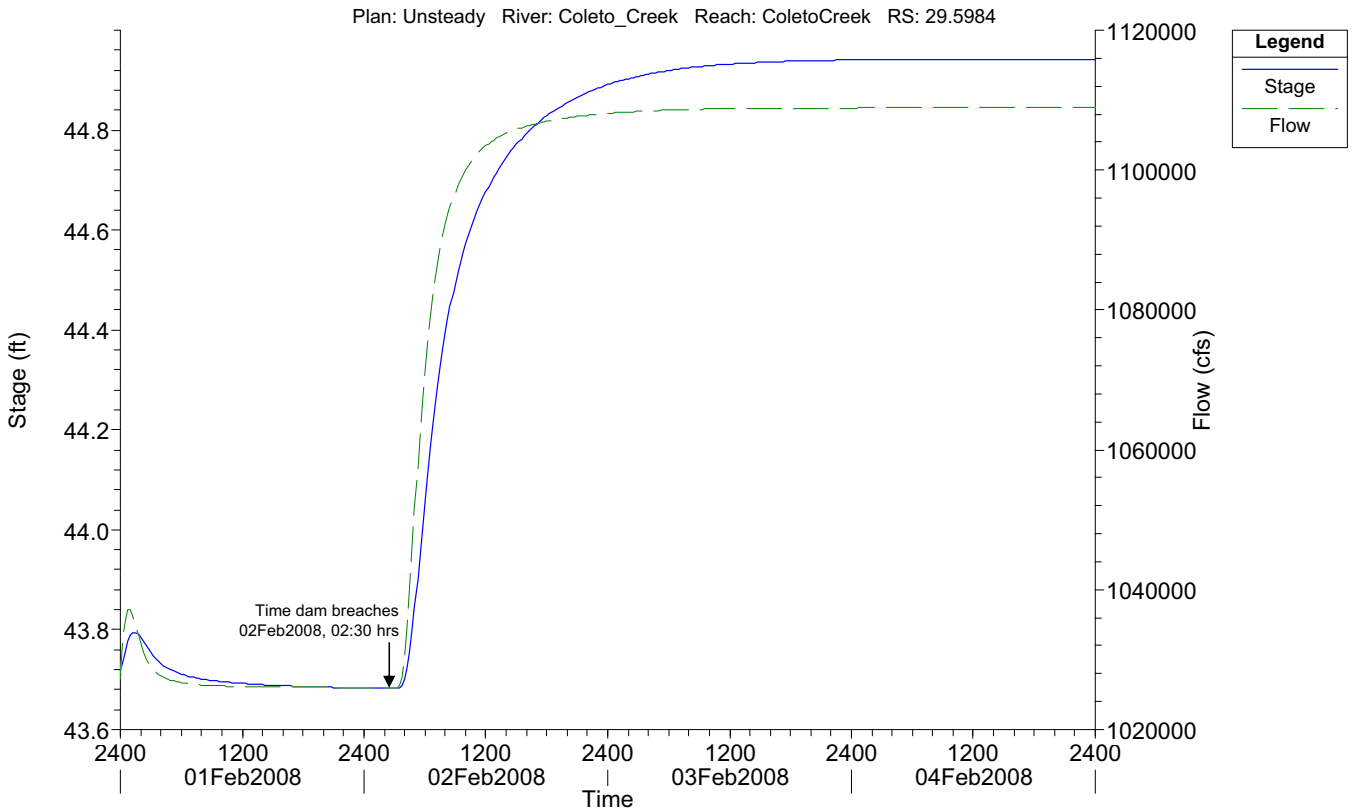

The legend "Bank Sta" represents the limit of the channel left and right banks in the model. Elevations in NAVD 88.

**Figure 2.4.4-10 Coleto Creek Model Cross Section at about 8.7 River Miles Upstream of the Confluence with the Guadalupe River**



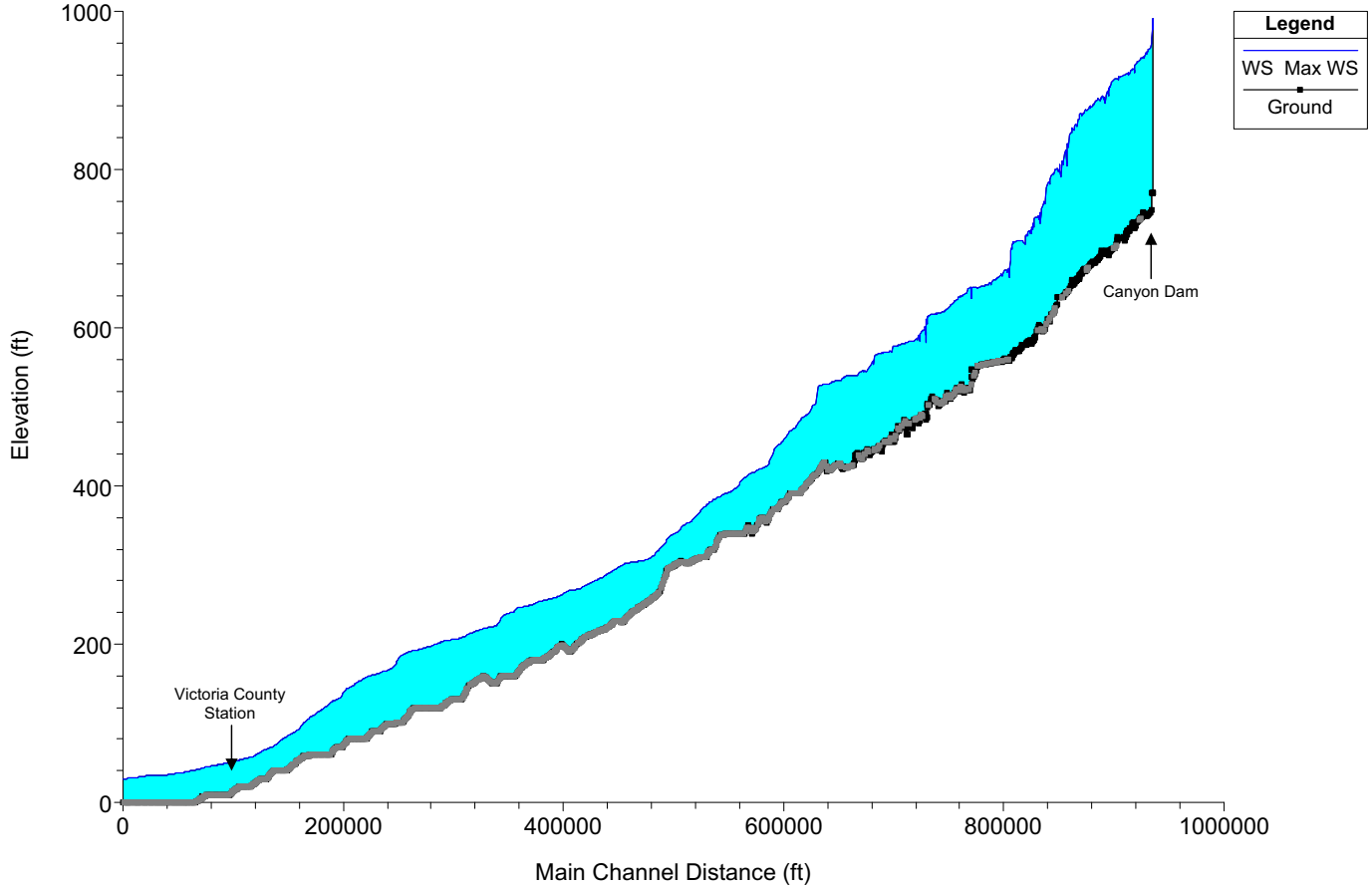

Note: Model simulation and dam breach begin at January 01, 2008 at 0:00 hours (selected arbitrarily).

**Figure 2.4.4-11 Water Level (NAVD 88) and Flow Hydrographs in the Guadalupe River near VCS for the Canyon Dam Breach Case**



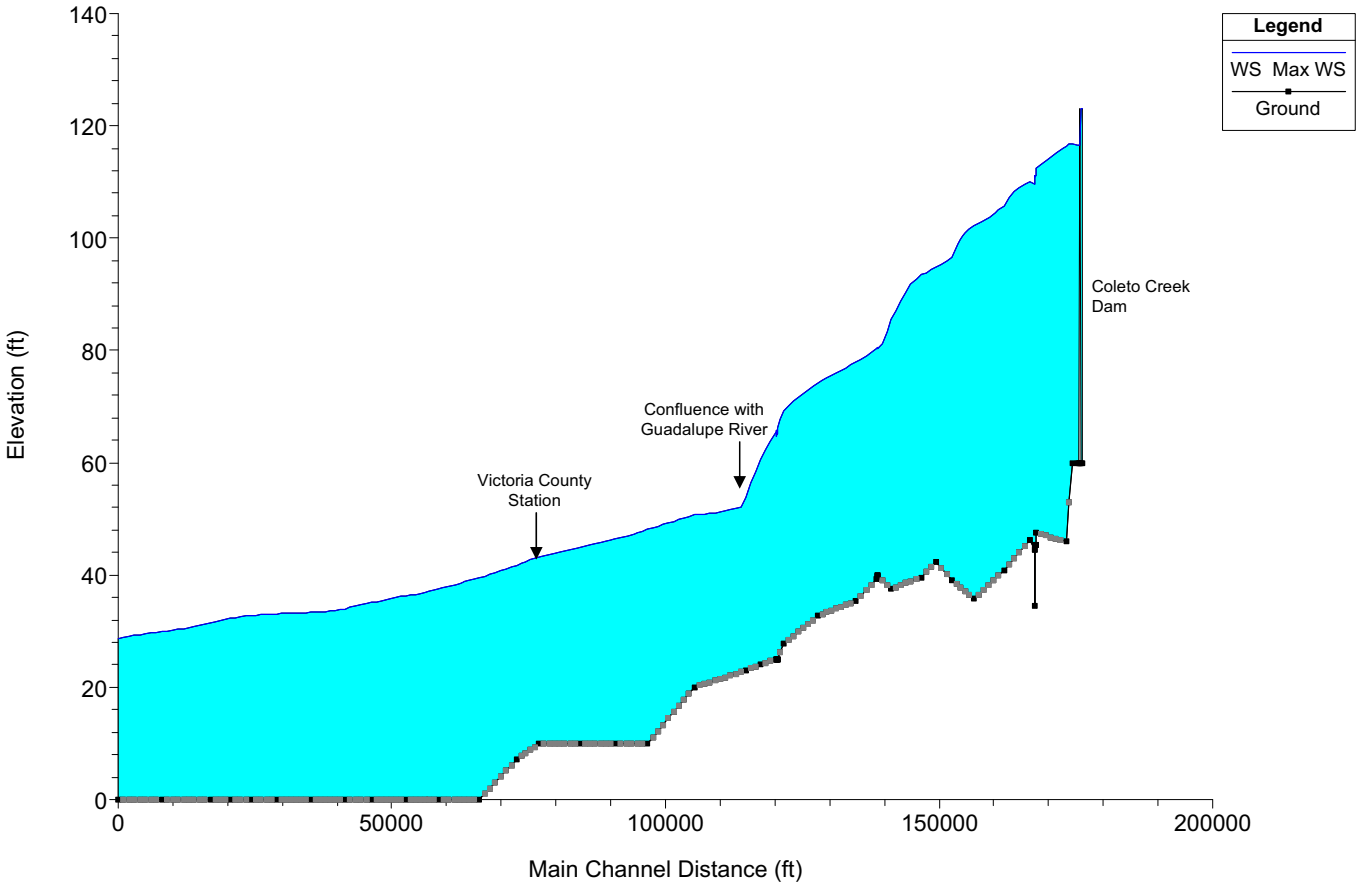

Note: Model simulation begins at 01-February-2008 and was selected arbitrarily.

**Figure 2.4.4-12 Water Level (NAVD 88) and Flow Hydrographs in the Guadalupe River near VCS for the Coleto Creek Dam Breach Case**




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

**Figure 2.4.4-13 Maximum Water Level Profile (NAVD 88) for the Canyon Dam Breach Case**




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

**Figure 2.4.4-14 Maximum Water Level Profile (in NAVD 88) for the Coleto Creek Dam Breach Case**

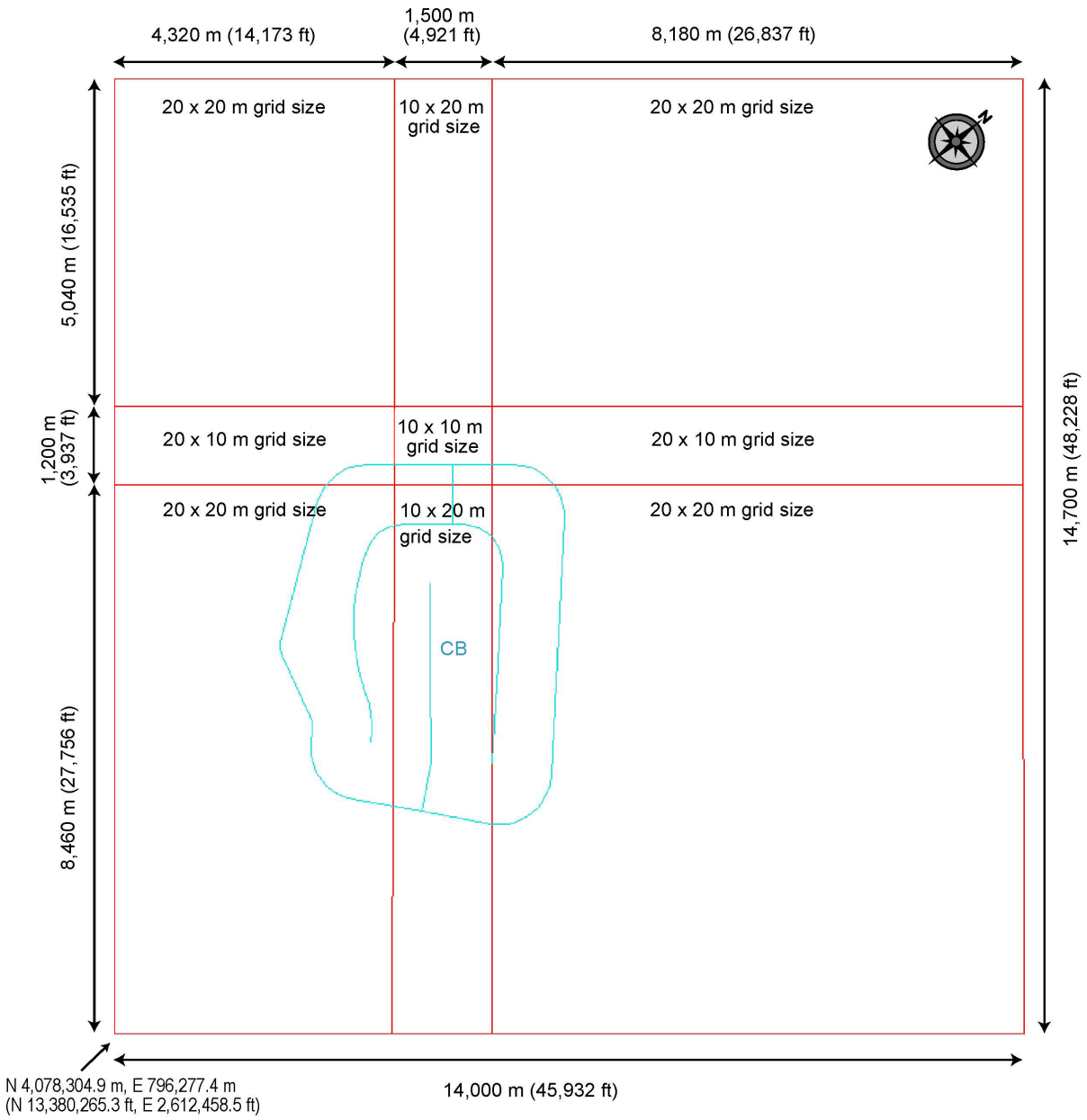



**Figure 2.4.4-15 Model Domain and Grid Sizes for the Cooling Basin Breach Model**

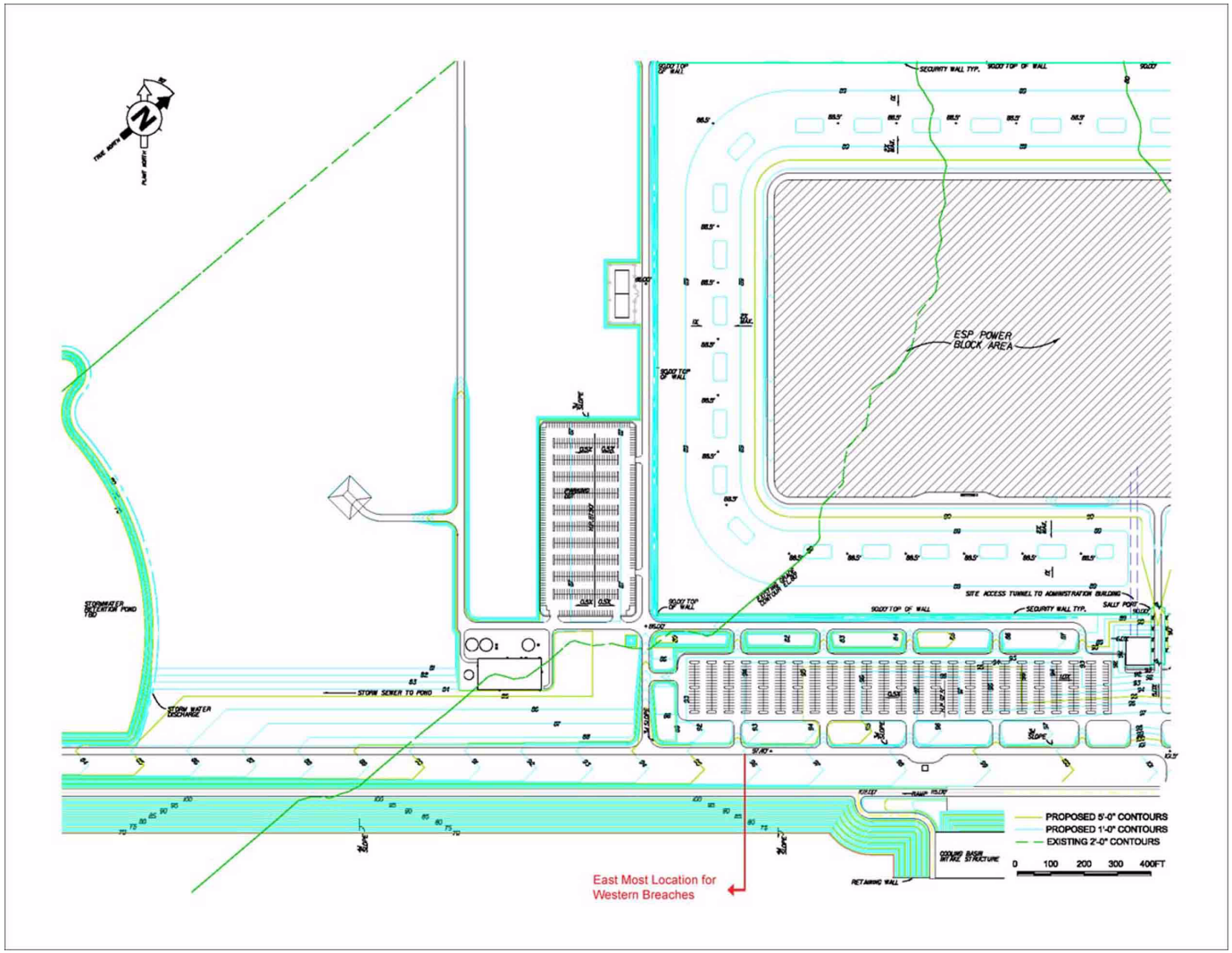



**Figure 2.4.4-16 Finish Grading West of the Power Block and Location Where Western Breach Begins**

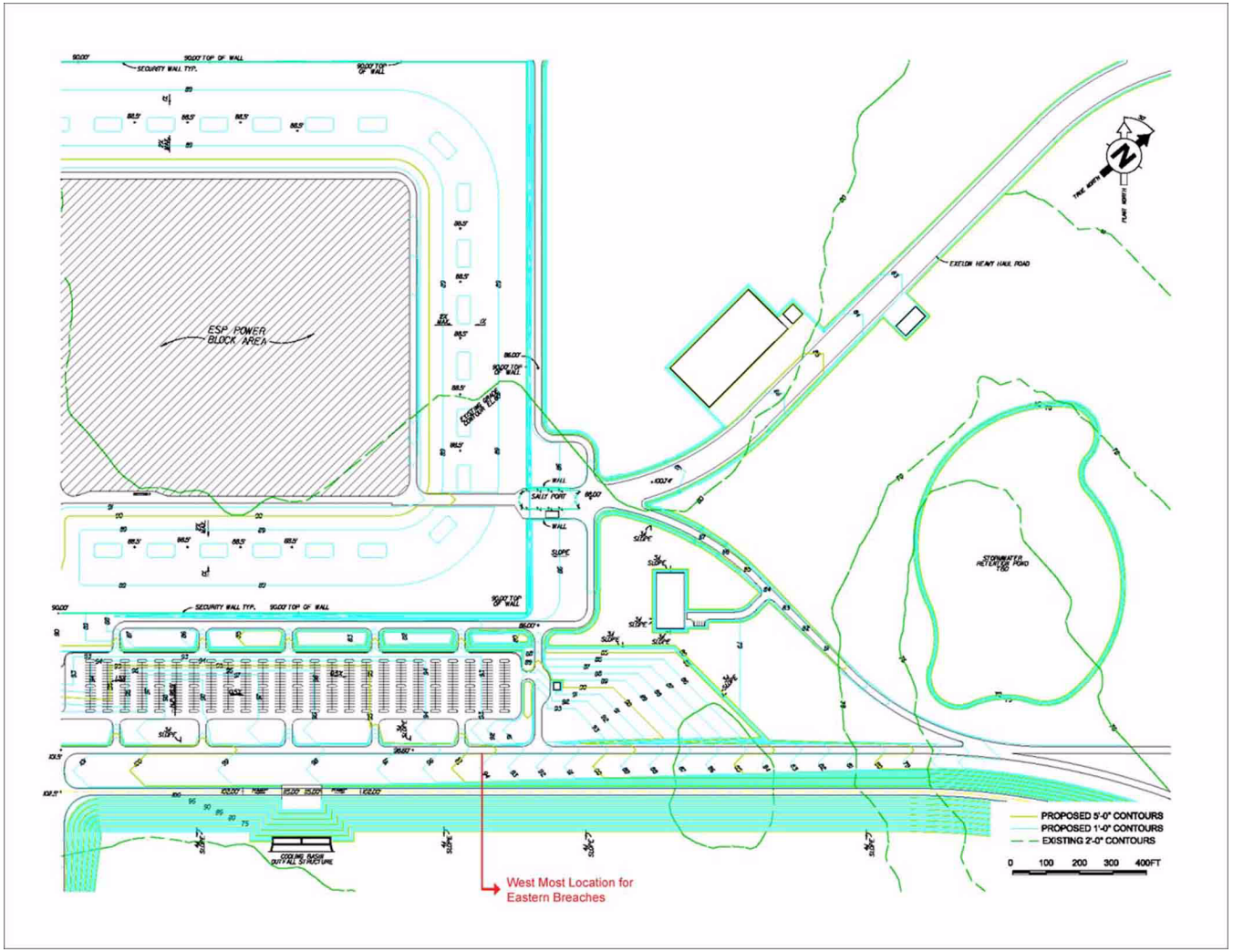
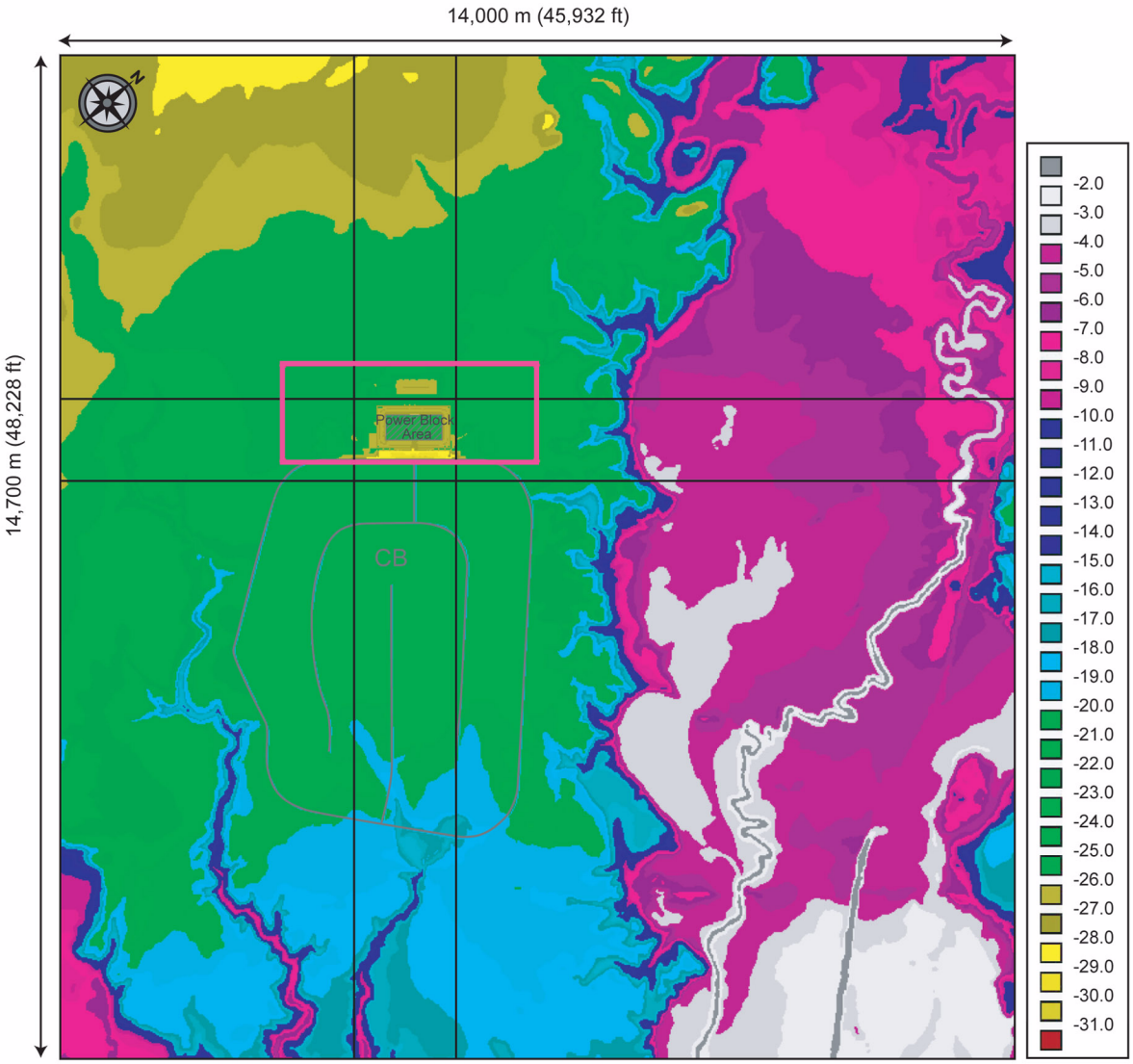
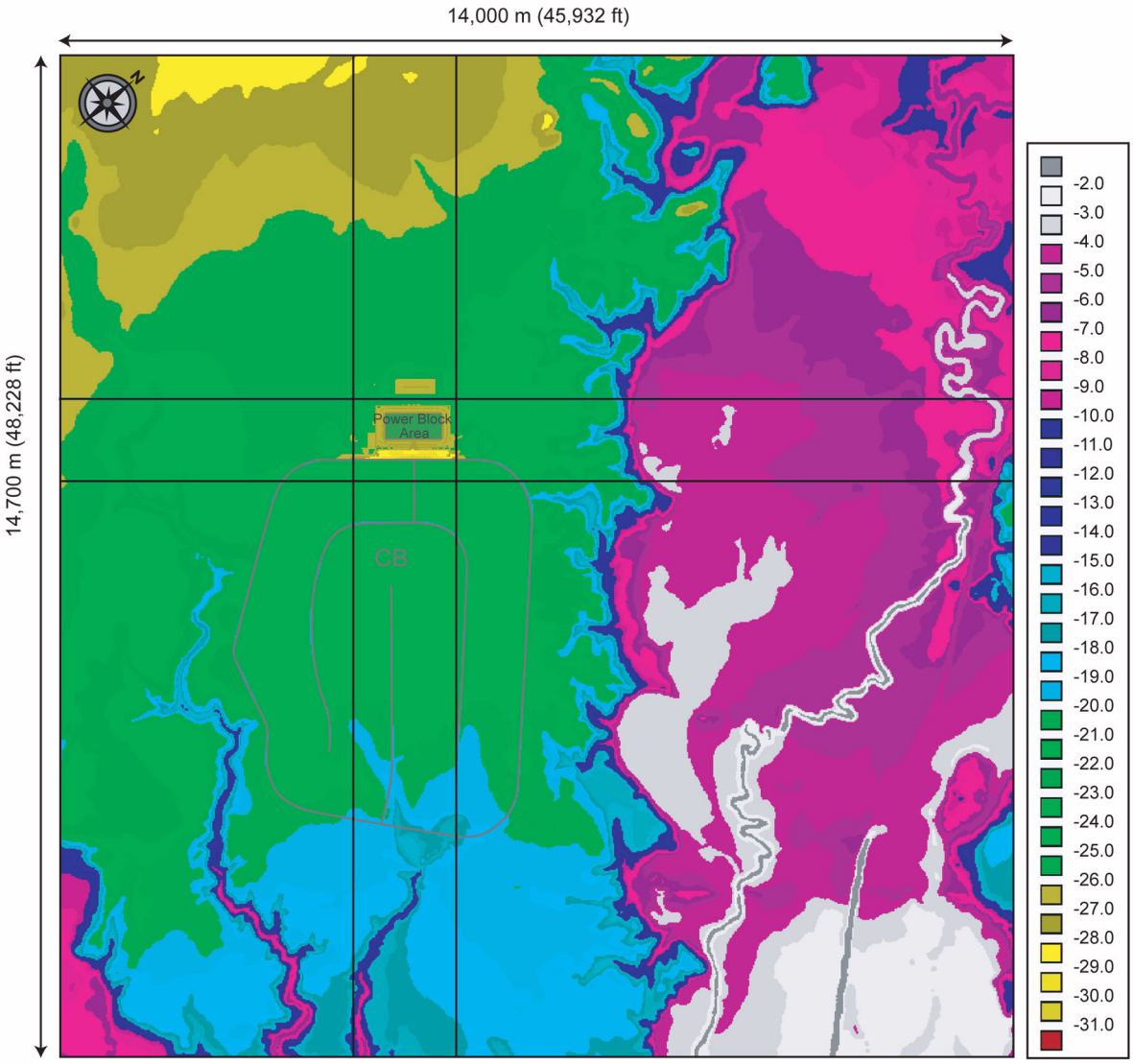



**Figure 2.4.4-17 Finish Grading East of the Power Block and Location Where Eastern Breach Begins**




Note: Black lines are boundaries of areas with the same model grid sizes and elevations above the NAVD 88 datum are represented as negative values in the model.

**Figure 2.4.4-18 Boundary Between the Power Block Area Finish Grade Data and USGS NED Data Used to Build the Model Bathymetry (Outlined by the Red Box)**




Note: Elevations above the NAVD 88 datum are represented as negative values in the model.

**Figure 2.4.4-19 Model Bathymetry in Meters, NAVD 88**


Note: Elevations above the NAVD 88 datum are represented as negative values in the model.

**Figure 2.4.4-20 Model Bathymetry near the Power Block Area in Meters, NAVD 88**

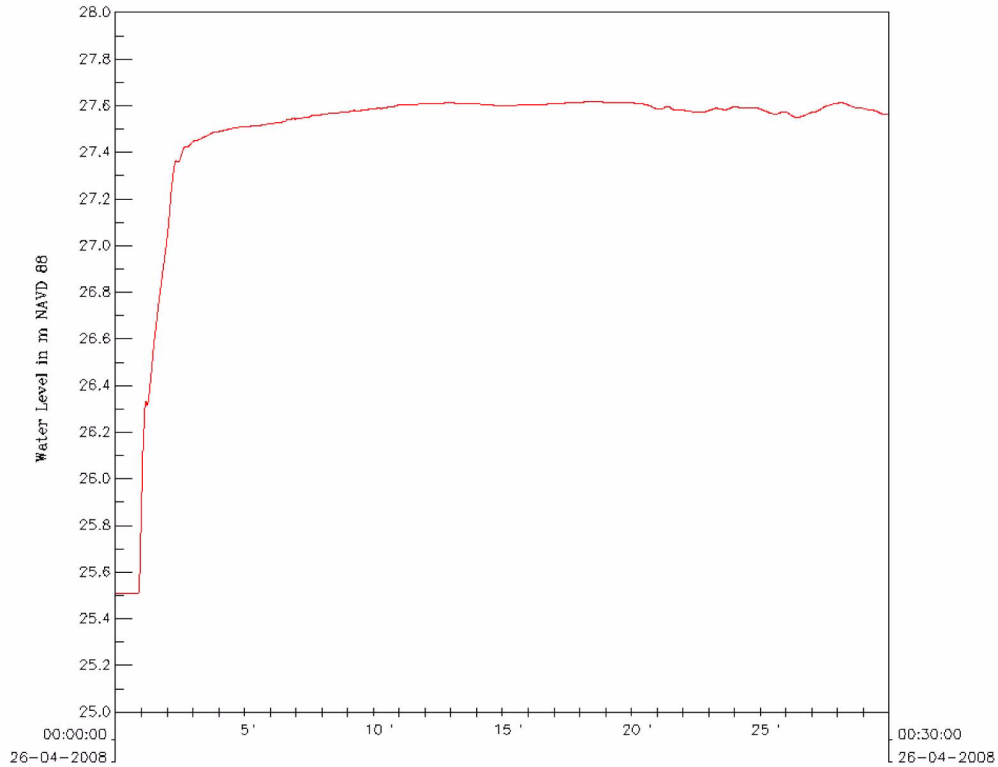



**Figure 2.4.4-21 Water Level Time Series at the Southwestern Corner of the Security Wall**

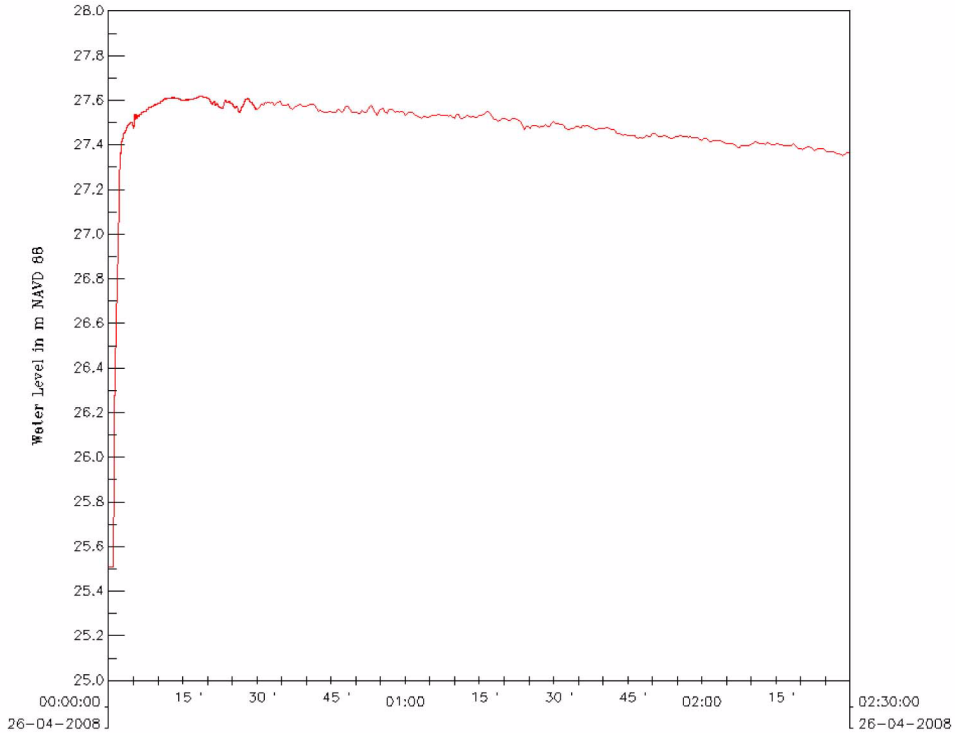



**Figure 2.4.4-21a Water Level Time Series at the Southwestern Corner of the Security Wall (Water Level for the First 30 Minutes is Based on Time Step of 0.01 Minutes)**

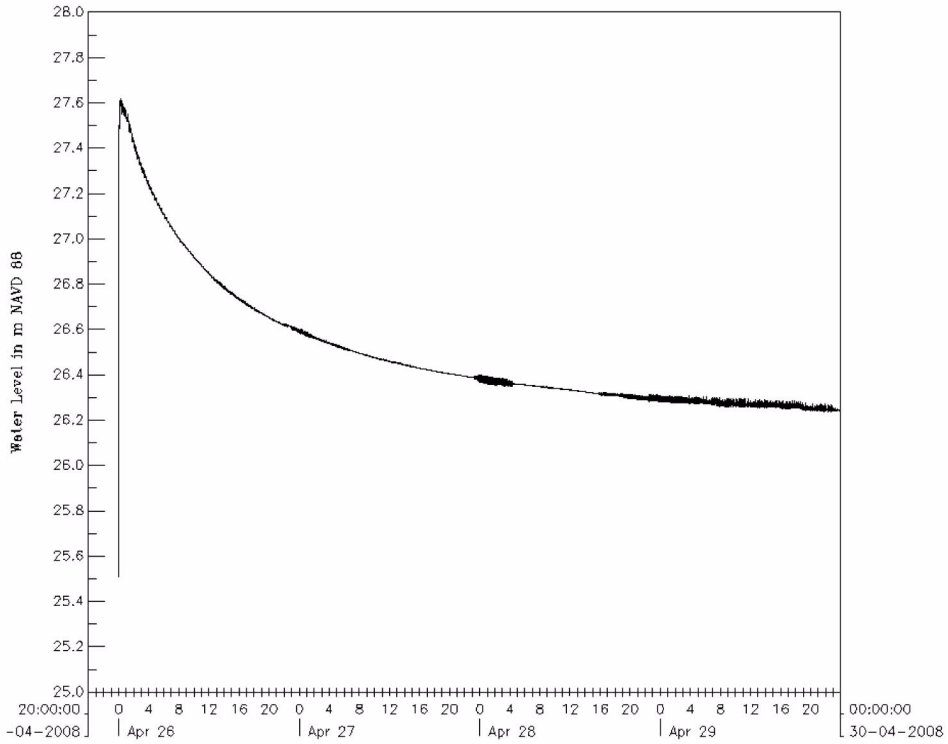



**Figure 2.4.4-21b Extended Time History of Water Level at the Power Block Where the Maximum Flood Occurs**

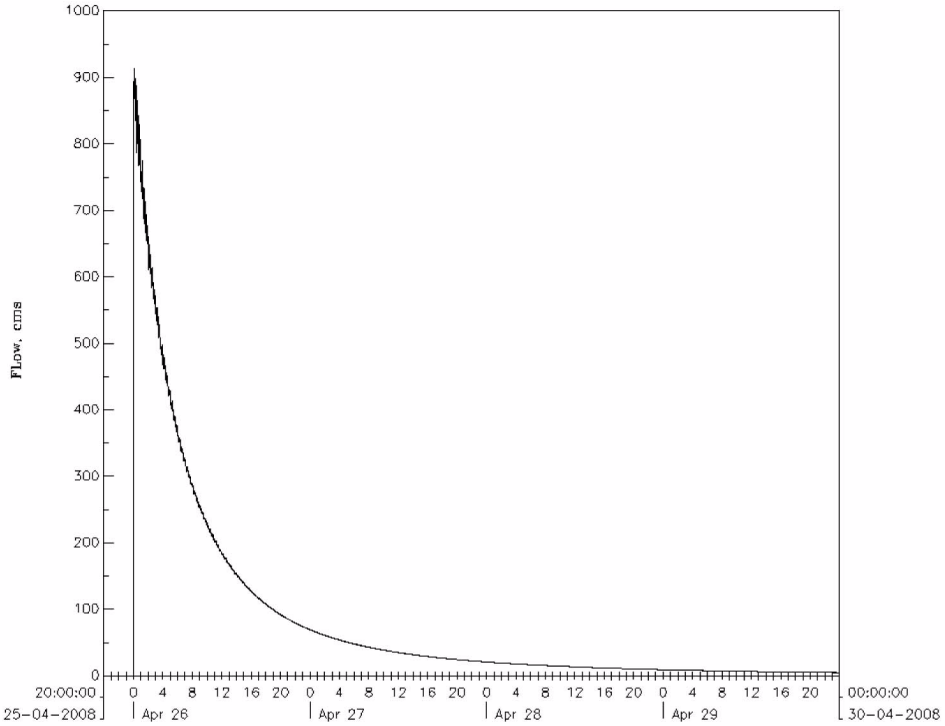



**Figure 2.4.4-21c Extended Time History of Breaching Flow from the Cooling Basin**

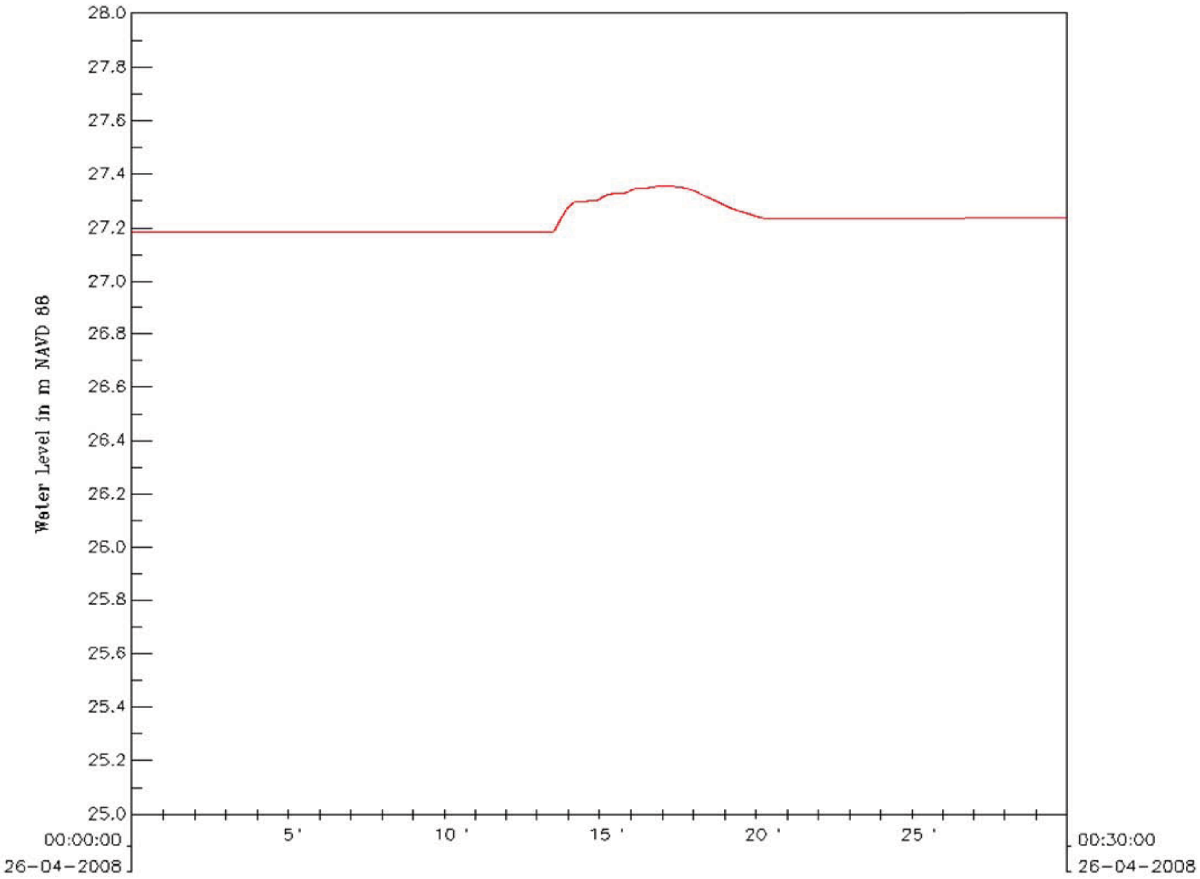



**Figure 2.4.4-22 Water Level Time Series Near the Midsection of the Security Wall on the South Side**

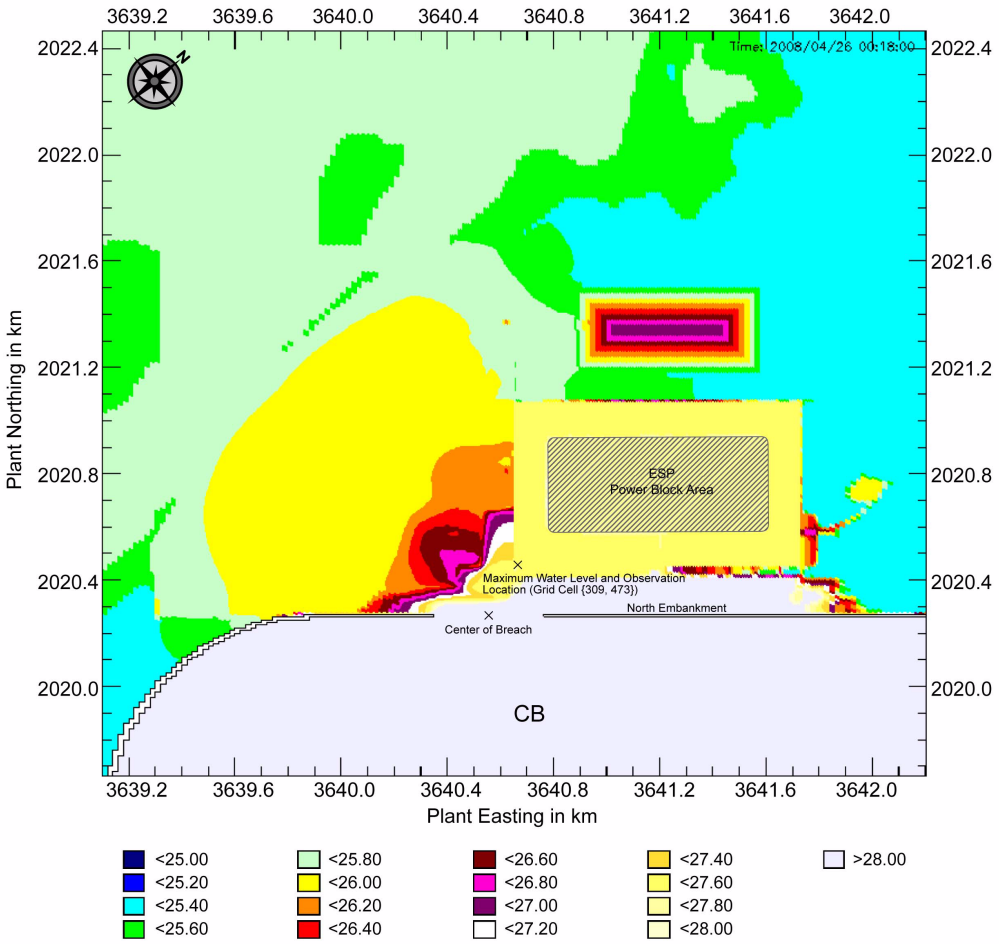



**Figure 2.4.4-23 Water Level Contours (in meters, NAVD 88) after 18 Minutes of the Western Cooling Basin Breach**

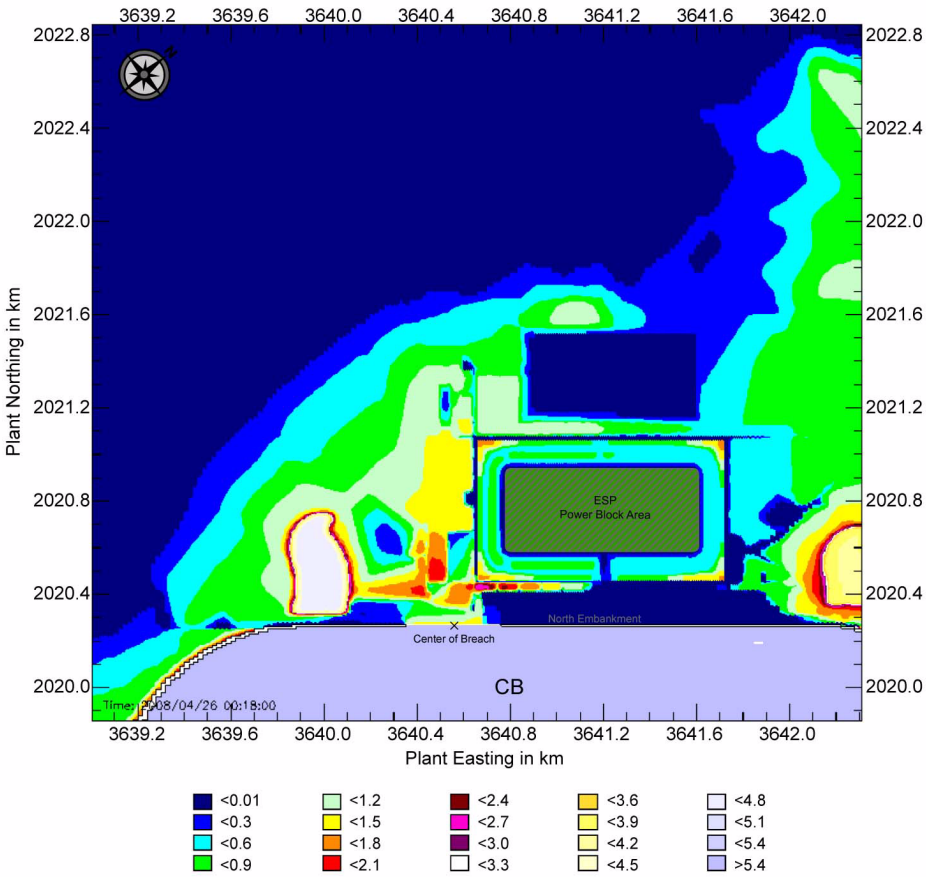



**Figure 2.4.4-24 Water Depth Contours (in meters) after 18 Minutes of the Western Cooling Basin Breach**

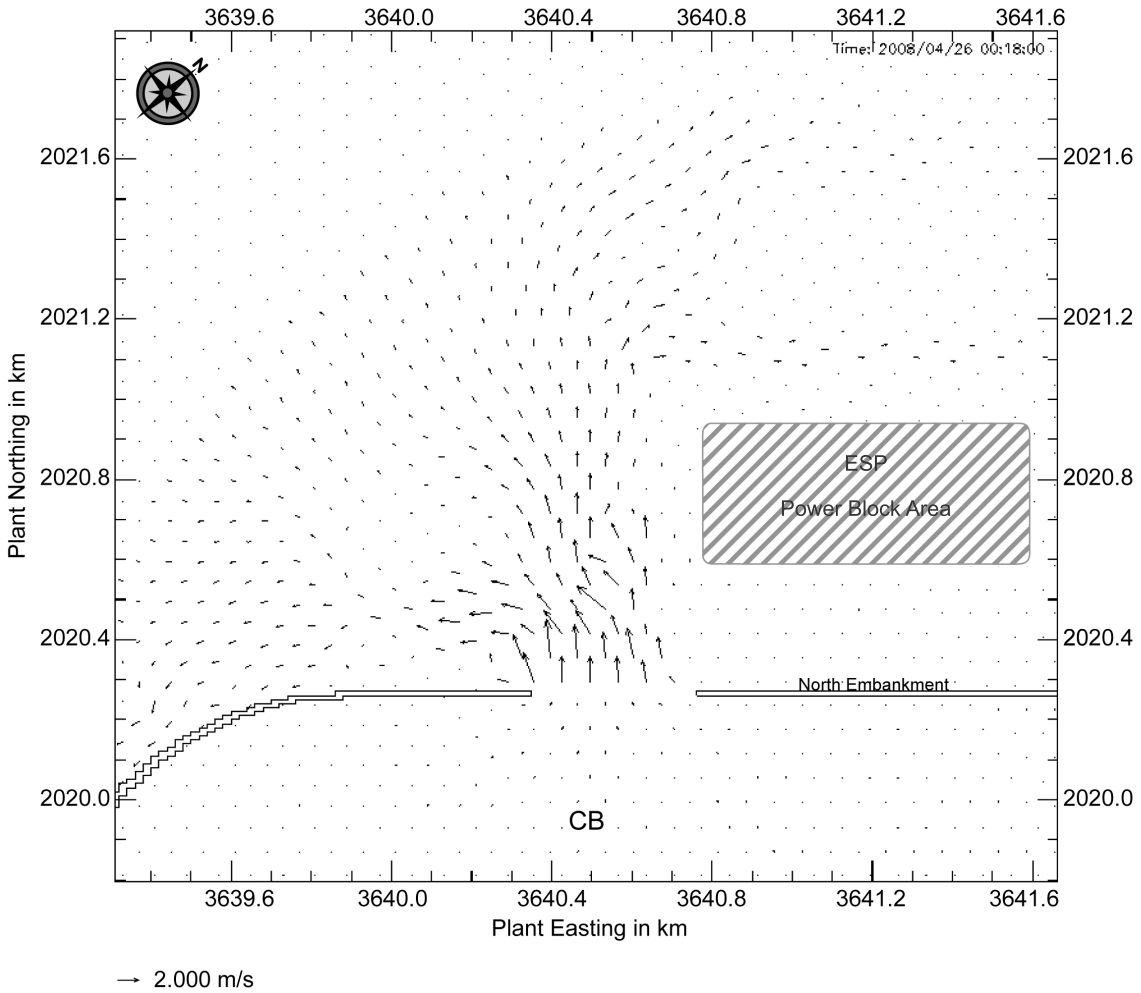



**Figure 2.4.4-25 Velocity Vectors after 18 Minutes of the Western Cooling Basin Breach**

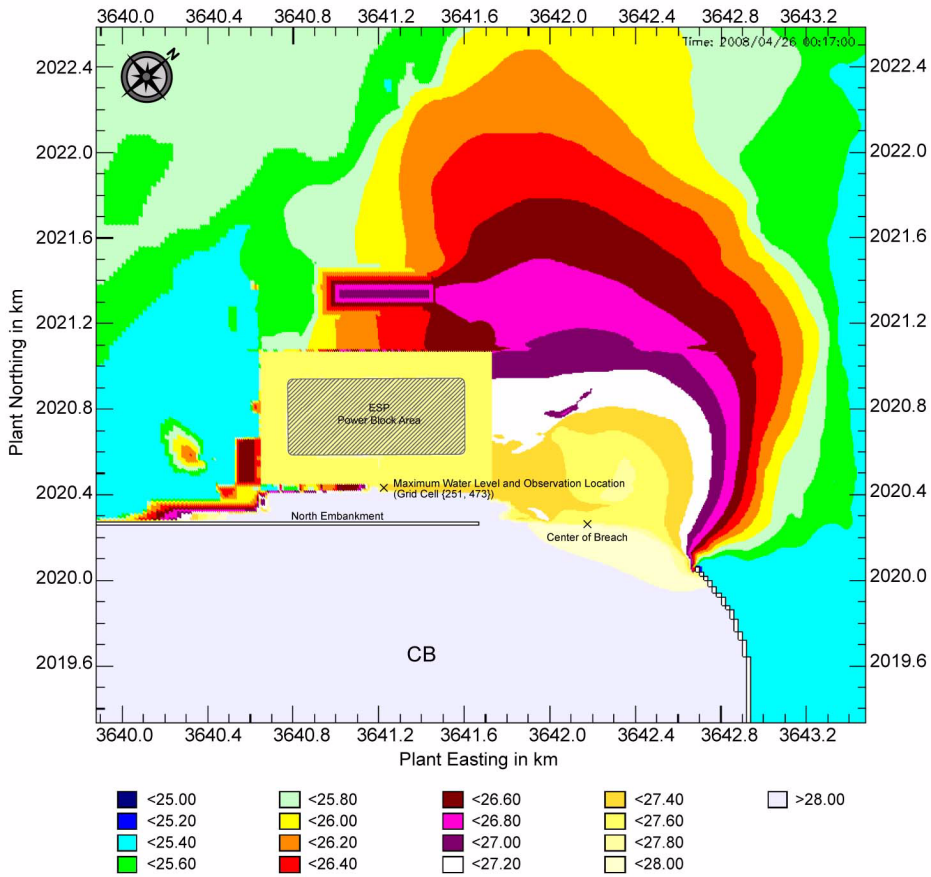



**Figure 2.4.4-26 Water Level Contours (in meters, NAVD 88) after 17 Minutes of the Eastern Cooling Basin Breach**

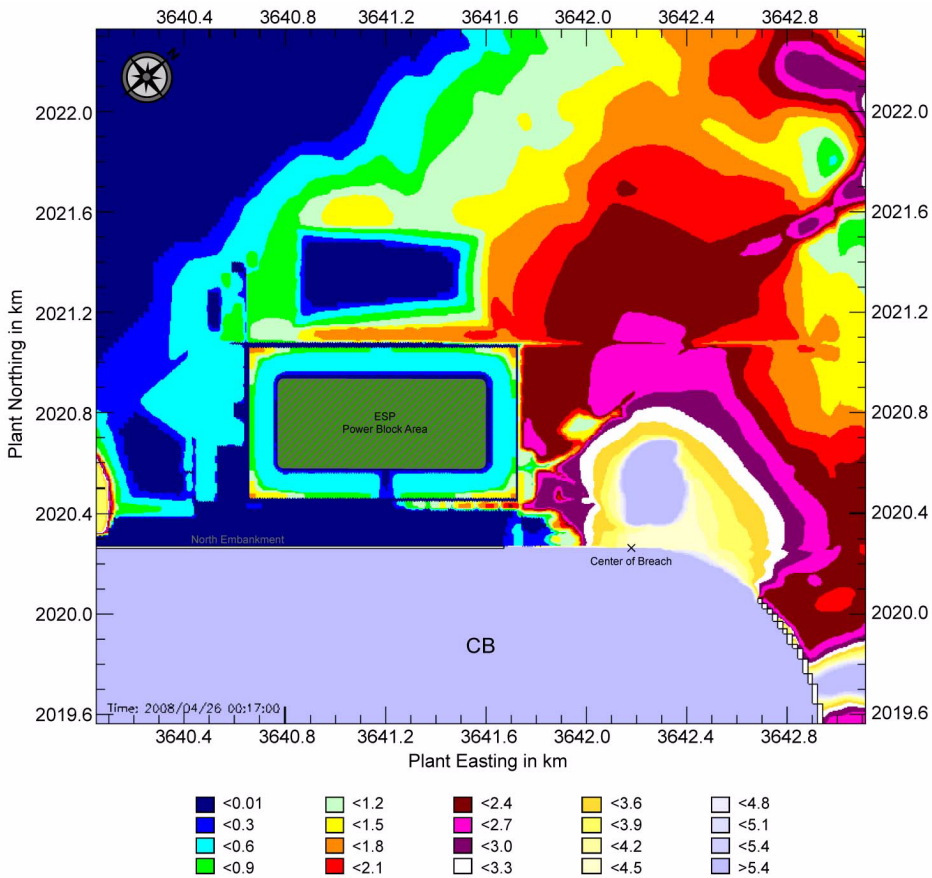



**Figure 2.4.4-27 Water Depth Contours (in meters) after 17 Minutes of the Eastern Cooling Basin Breach**

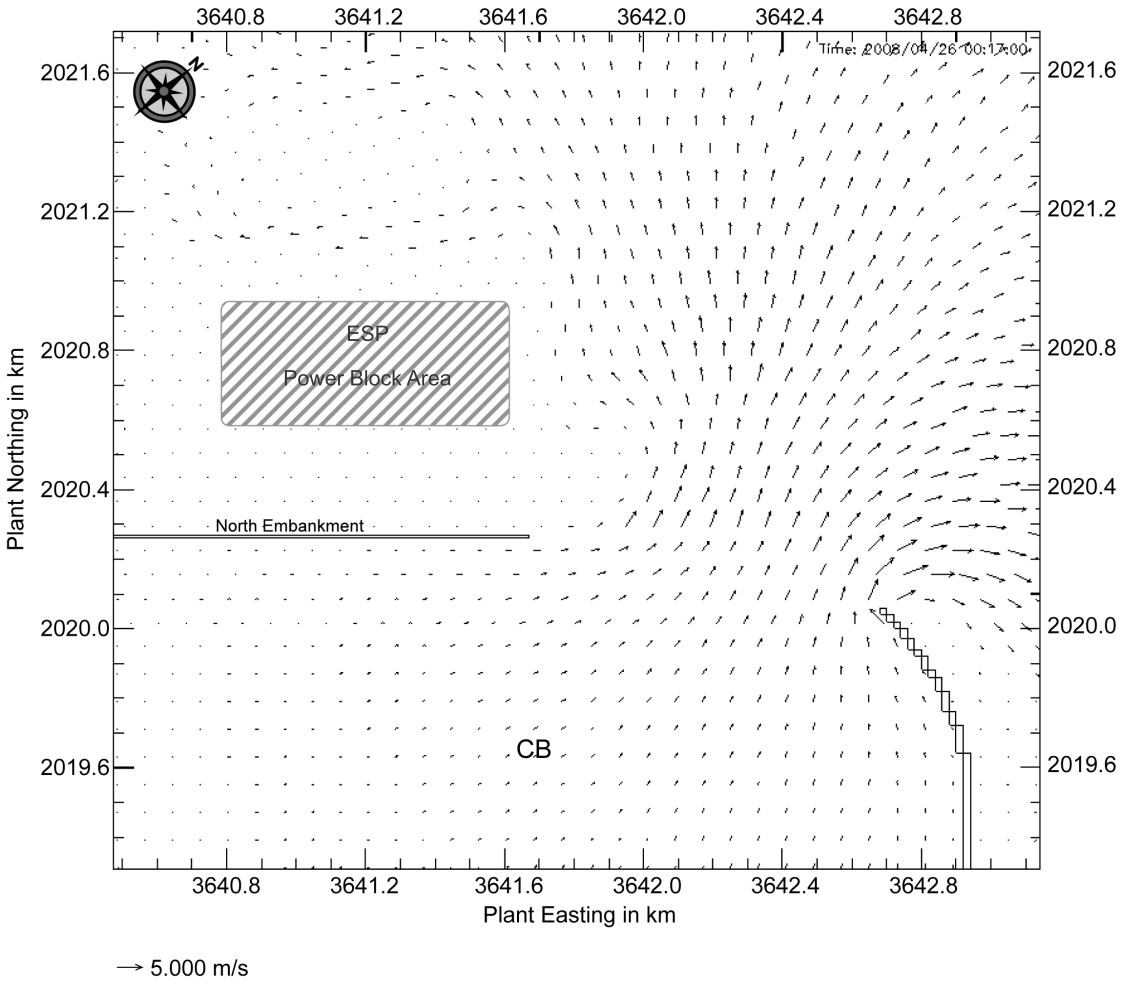



**Figure 2.4.4-28 Velocity Vectors after 17 Minutes of the Eastern Cooling Basin Breach**