2.4S.4 Potential Dam Failures

The following site-specific supplement addresses COL License Information Items 2.14 and 3.5.

This section addresses the SRP Section 2.4.4 Acceptance Criteria Limits from the reference Table 2.1-1, which states that the flood level from failure of existing and potential upstream or downstream water control structures will not exceed 30.5 cm (1.0 ft) below grade. The nominal plant grade for the safety facilities of STP 3 & 4 is 34.0 ft mean sea level (MSL) and the design entrance level slab elevation is 35.0 ft MSL. The flooding level at STP 3 & 4 resulting from the worst case dam failure scenario, the postulated MCR breach was estimated to be 47.6 ft MSL, exceeding the reference ABWR DCD site parameter flood level criteria. The departure from the DCD site parameter flood level and the evaluation summary are documented in STP DEP T1 5.0-1. Subsection 2.4S.4 develops the flooding design basis for considering potential hazards to the safety-related facilities due to potential dam failures.

The STP 3 & 4 site is located on the west bank of the Colorado River in Matagorda County, Texas, about 10.5 river miles upstream of the Gulf Intracoastal Waterway (GIWW). There are a total of 68 dams with storage capacity in excess of 5000 acre-feet (AF) on the Colorado River and its tributaries upstream of the STP site. These dams and reservoirs are owned and operated by different entities including the Lower Colorado River Authority (LCRA), the U.S. Bureau of Reclamation (USBR), the Colorado River Municipal Water District (CRMWD), other local municipalities and utilities. Figures 2.4S.4-1(a) and 2.4S.4-1(b) show the locations of the 68 dams. Specific information of these dams that are relevant to the flood risk assessment of STP 3 & 4 is summarized in Table 2.4S.4-1, based on data collected primarily from the Texas Water Development Board (TWDB), Texas Commission for Environmental Quality (TCEQ), and LCRA. The six hydroelectric dams – Buchanan, Roy Inks, Alvin Wirtz, Max Starcke, Mansfield, and Tom Miller, owned and operated by LCRA are known as the Highland Lake dams.

In Texas, both private and public dams are monitored and regulated by TCEQ under the Dam Safety Program. Existing dams, as defined in Rule §299.1 Title 30 of the Texas Administrative Code (Reference 2.4S.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) on Hydrologic Criteria for Dams are the minimum acceptable spillway evaluation flood (SEF) for re-evaluating dam and spillway capacity for existing dams to determine whether upgrading is required. Similarly, on the structural considerations, evaluation of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems such as seepage, cracks, slides, conduit and control malfunctions, and other structural and maintenance deficiencies which could lead to failure of a structure.

Following the 1987 National Dam Safety Inspection Program recommendations of the Texas Water Commission, a predecessor agency of the TCEQ, to upgrade two of the Highland Lake dams due to unsafe condition, LCRA initiated a program to evaluate all six Highland Lake dams with respect to hydrologic, structural and geotechnical criteria. In 1990, LCRA began a 15-year plan of Dam Modernization Program to address the safety condition of five of the six dams. A 1992 dam safety evaluation study commissioned by LCRA (Reference 2.4S.4-2) indicates that Wirtz, Starcke, and Tom Miller Dams would be overtopped during a Probable Maximum Flood (PMF) event, and certain sections of Buchanan, Wirtz, and Tom Miller Dams could have instability problems during severe flood conditions. The concrete dam sections of Mansfield Dam, however, would be stable during the PMF. At the completion of LCRA's Dam Modernization Program in January of 2005, substantial upgrade work had been undertaken at Buchanan, Inks, Wirtz, and Tom Miller Dams to address the unsafe conditions (Reference 2.4S.4-3). Upgrade at Mansfield Dam was considered not necessary as it is able to withstand the PMF without further reinforcement. Even in the event of failures of either Buchanan, Inks, Wirtz, or Starcke dams, Mansfield Dam would hold their flood volumes without overtopping (Reference 2.4S.4-4).

The UFSAR of STP 1 & 2 (Reference 2.4S.4-5) identifies two dam failure scenarios that are most critical to the flooding at the STP site. They are: (1) the breaching of the embankment of the onsite Main Cooling Reservoir (MCR); and (2) the postulated cascade failure of the major upstream dams on the Colorado River. These two scenarios also form the basis of the maximum flood level evaluation for STP 3 & 4 resulting from potential dam failures because the watershed and topographic conditions remain relatively unchanged since the preparation of the UFSAR for STP 1 & 2, and also because there are no new dams (including the previously proposed Columbus Bend Dam) planned for the Colorado River in the next 50 years, according to the 2007 State Water Plan (Reference 2.4S.3-6, also discussed in Subsection 2.4S.3.4.2) The dam failure scenarios and the postulated flood risk are discussed further in the following subsections.

2.4S.4.1 Dam Failure Permutations

2.4S.4.1.1 Failures of Upstream Dams on the Colorado River

Of all the dams on the Colorado River upstream of the STP 3 & 4 site, Mansfield Dam would generate the most significant dam break flood risk on the site. Mansfield Dam has the largest dam height of 266.4 ft and the largest reservoir storage capacity of 3.3 million acre-feet (MAF), at top of the dam. Among all the dams upstream, Mansfield Dam is also closest to the site at about 305 river miles upstream of the STP 3 & 4 site. The next major dam upstream that could pose significant flood risk to the site is the Buchanan Dam located at about 402 river miles upstream of STP 3 & 4. It has a height of 145.5 ft and a top-of-dam storage capacity of 1.18 MAF. Further upstream, the Simon Freese Dam, with a height of 148 ft and a top-of-dam storage capacity of 1.47 MAF, and the Twin Buttes Dam, with a height of 134 ft and top-of-dam storage capacity of 1.29 MAF are considered to have major, though not as significant, contribution to the flood risk at the STP site. They are located at about 199 miles and 290 miles, respectively, upstream of Buchanan Dam.

There are two failure permutations postulated of the upstream dams:

- Scenario No. 1 – Simultaneous failure of all upstream dams induced by a seismic event. The failure is to occur coincidentally with a 2-year design wind event and a

500-year flood or a one-half probable maximum flood (PMF) per American National Standard ANSI/ANS-2.8 (Reference 2.4S.4-7).

- Scenario No. 2 – Domino-type failure of upstream dams with the same coincidental wind and flood events as in Scenario No. 1. It is postulated that the upstream-most dam(s) would fail first, thereby releasing a dam break flood wave (or waves) that propagates downstream and triggers the failure of the downstream dams one after another in a cascading manner. It is assumed that the 56 dams on the Colorado River and its tributaries upstream of Buchanan Dam (with top-of-dam capacity over 5000 AF) would fail in such a manner that their flood flow, expressed in terms of their respective top-of-dam storage volumes, would arrive at Lake Buchanan at approximately the same time, triggering the failure of Buchanan Dam. The dam break flood flow from Buchanan Dam would then propagate downstream to Lake Travis, overtopping Mansfield Dam and causing it to fail. The dam break flood from Mansfield Dam then propagates downstream to the STP 3 & 4 site. The failure is to occur coincidentally with a 2-year design wind event and a 500-year flood or a one-half probable maximum flood (PMF) per American National Standard ANSI/ANS-2.8 (Reference 2.4S.4-7).

Three upstream dams, Inks, Wirtz, and Starcke, located between Buchanan and Mansfield Dams, and two other upstream dams, Tom Miller and Longhorn Dams, located at 20 miles and 27 miles downstream of Mansfield Dam, were not included in the dam break analysis as their dam heights and potential flood volumes would have insignificant impact on the flood risk as compared to Mansfield Dam or Buchanan Dam.

There are five “off-channel” dams located on the tributaries of the Colorado River between Mansfield Dam and the STP site. They are: Decker Creek Dam (Lake Long), Bastrop Dam, Cummins Creek WS SCS Site 1 Dam, Cedar Creek Dam (Fayette Reservoir), and Eagle Lake Dam. These off-channel storage dams were also assumed to have no effect on the maximum dam break flood level at the STP 3 & 4 site, as compared to the major dams on the main stem of the Colorado River.

Of these two permutations, Scenario No. 2 would generate the most critical flood level at STP 3 & 4 because of the deliberate alignment of the travel and arrival of the dam breach flood volumes and flood peaks from the major upstream dams. Consequently, only the flood risk resulting from Scenario No. 2 was further evaluated.

Upstream dam failures induced by hydrologic causes such as probable maximum flood (PMF) will not be the controlling scenario in the evaluation of the maximum flood risk at the STP site. This is because the large dams with high hazard potential, such as O.C. Fischer, Simon Freese, Buchanan and Mansfield Dams, as listed in Table 2.4S.1-1, were either designed or have been upgraded 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 2.4S.4-1). Mansfield Dam, in particular, would be able to hold the dam break flood volumes of either Buchanan, Wirtz, or Starcke Dams. Besides, the assumption that a domino-type dam failure of the 56 dams upstream of Buchanan with an aggregated top-of-dam storage volume of 6.87 MAF all arriving at Buchanan at about the same time is highly conservative and would have bounded the potential flood risk caused by hydrological dam failures.

2.4S.4.1.2 Postulated Failure of the Main Cooling Reservoir

The MCR is enclosed by a rolled-earthen embankment, rising an average of 40 ft above the natural ground surface south of the plant site. The centerline of the north embankment is approximately 2340 ft south of the centerline of the reactor buildings of STP 3 & 4. Site grade near the northern embankment is in the range of El. 27 ft MSL to El. 29 ft MSL, and the top of the embankment is at about El. 65.75 ft MSL. Normal maximum operating level of the reservoir is at El. 49.0 ft MSL, which is about 20 to 22 ft higher than the site grade near the northern embankment. Postulated failure mechanisms of the earth embankment includes 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 due to overtopping from flood or wind-wave events.

As discussed in the STP 1 & 2 UFSAR (Reference 2.4S.4-5), failure of the MCR embankment due to any of these probable 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. The most conservative conjecture of such a failure suggested that an embankment section of several hundred feet long would translate downstream several tens of feet off of its original location (Reference 2.4S.4-5). This failure scenario was modeled using a 2-dimensional flood model as described in STP 1 & 2 UFSAR by assuming an instantaneous removal of a 400-ft long section of the embankment. In order to ensure sufficient freeboard in the design of the safety related facilities for flood protection, the postulated breach length was further increased from 400 ft to 4000 ft, incrementally, to determine the most critical flooding impact to the site. A 2000-ft or wider breach was found to produce the highest flood level at the safety facilities of STP 1 & 2.

A similar approach was used for STP 3 & 4 by varying the breach length in an effort to predict the maximum flood level that would be experienced by STP 3 & 4 safety related facilities as a result of the highly improbable MCR failure event.

2.4S.4.1.3 Potential for Landslide and Waterborne Missiles

The potential for major scale landslide, and hence blockage of streams on the Lower Colorado River in the vicinity of the STP site, is highly improbable due to the flat terrain. This is consistent with the conclusion of the UFSAR for STP 1 & 2 (Reference 2.4S.4-5). According to the investigation, there is no threat posed to the STP site due to surge from bank material sliding into the Lower Colorado River.

The potential for waterborne missiles reaching the STP site due to upstream dam failure is not considered to be critical because the site is located in the flood plain of the Lower Colorado River where the flood flow velocities are in general substantially lower than that in the main channel. Although there is a potential for waterborne missiles due to the MCR's breach, these missiles are not considered to be critical to the design of the safety related structures compared to tornado missiles. The static and dynamic effects of the MCR breach on the plant structures are discussed in Section 3.4.

2.4S.4.2 Unsteady Flow Analysis of Potential Dam Failures

2.4S.4.2.1 Colorado River Dams

The dams on the Colorado River are discussed in Subsection 2.4S.4.1. Table 2.4S.4-1 lists the height, length, top-of-dam storage capacity, type, and year of completion of the 68 dams with a top-of-dam storage capacity larger than 5000 AF each. Of these 68 dams, Mansfield Dam, Buchanan Dam and 56 other dams upstream of Buchanan Dam were selected for inclusion in the dam break analysis. Dams with less than 5000 AF storage capacity, i.e., less than 0.2% of that of Mansfield Dam, were excluded from further evaluation as the impact of their potential breaching on the flood risk at the site would be minimal. The top-of-dam storage volume of Mansfield Dam is about 3.3 MAF, estimated from the elevation-storage capacity curves given in Reference 2.4S.4-8. Similarly, the top-of-dam storage volume of Buchanan Dam is estimated to be about 1.18 MAF. The combined top-of-dam-storage volume of the 56 dams upstream of Buchanan Dam is 6.87 MAF.

2.4S.4.2.1.1 Conceptual Unsteady Flow Analytical Model

The dam breach option of the USACE River Analysis System computer program (HEC-RAS) Version 3.1.3 (Reference 2.4S.4-9) was used to simulate the dam breach flood waves, which were then routed downstream to the STP 3 & 4, using the unsteady flow option of the program.

In the conceptual dam break flood model, the 56 dams upstream of Buchanan Dam would fail in a domino manner, with their combined top-of-dam storage capacity, totaling 6.87 MAF, arriving at Buchanan Dam at approximately the same time. As the flood level at Buchanan Dam rises to about 3 ft over the dam crest elevation of 1025.35 ft MSL, the dam would fail, thereby releasing the flood storage of Buchanan Dam plus the combined flood volumes from the 56 upstream dams. In accordance with the combined events requirements stipulated in the American National Standard ANSI/ANS-2.8 (Reference 2.4S.4-7), the evaluation of potential flood risks as a result of non-hydrologic dam break failures should also consider a coincidental event equal to a 500-year flood or one-half probable maximum flood (PMF), whichever is less. In this analysis, a constant flood flow of 500,000 cfs, slightly higher than the peak Standard Project Flood (SPF) inflow at Buchanan Dam and the 500-year flood peak inflow at Mansfield Dam, was conservatively used to represent the coincidental flow. The SPF and 500-year flood flow at several locations on the Colorado River are listed in Table 2.4S.4-2. They were estimated by Halff Associates, Inc. as part of the Lower Colorado River flood damage evaluation project conducted for LCRA and Fort Worth District Army Corps of Engineer (Reference 2.4S.4-10). The 500,000 cfs coincidental flow was applied to the entire model reach from Buchanan Dam to the downstream boundary at 4600 ft (0.9 river miles) upstream of the Gulf Intracoastal Waterway.

The flood wave from the breaching of Buchanan Dam would propagate down to the 266.4-ft high Mansfield Dam, with a crest elevation at 754.1 ft MSL and a top-of-dam storage capacity of 3.30 MAF. (In 1941, a 4-ft parapet wall was added to the dam crest raising its elevation from 750.1 ft MSL to 754.1 ft MSL to provide additional flood storage capacity.) Mansfield Dam was postulated to fail when it was overtopped by 3 ft at El. 757.1 ft MSL. The three dams located between Buchanan and Mansfield Dams: Roy Inks, Alvin Wirtz, and Max Starcke Dams, have a combined storage of about 298,300 AF. These dams were not assumed to fail in the dam break model because their combined total storage amounts to only about 9% of the total dam break flood volume at Mansfield. The SPF flood hydrographs from 19 tributaries between Buchanan and Mansfield Dams as estimated by Halff Associates, Inc. in the flood damage evaluation study (Reference 2.4S.4-10) were included as tributary inflows to this reach. The tributary inflows together with the dam break flood wave from Mansfield Dam were then routed to the STP 3 & 4 site in the HEC-RAS model.

2.4S.4.2.1.2 Physical Dam Data and Estimates of Breached Sections

Buchanan Dam, located at about 402 river miles upstream of STP 3 & 4, is 10,987 ft in length. It has two separate multiple concrete arch sections as well as a number of gravity sections (Reference 2.4S.4-8). The main dam section consists of 29 concrete arches, each of 70 ft in width and 145.5 ft in height. The total length of this multiple concrete arch section is 2030 ft and it occupies the deepest part of the river channel. To the right (looking downstream) is another shorter multiple concrete arch section of 805 ft in length, consisting of 23 arches of 35 ft wide each. Following the guidelines from Federal Energy Regulatory Commission (FERC) on dam break analysis (Reference 2.4S.4-11), 15 of the 29 larger arches (70 ft wide each) and 12 of the 23 smaller arches (35 ft wide each) were assumed to breach in the simulation. The breach section in the model was represented by a vertical section with a total width of 1470 ft and extending from the top of the dam to the bottom. The time to complete the breach was assumed to be 0.1 hour, based on the guidelines from FERC for the estimation of the dam breach parameter (Reference 2.4S.4-11). The model cross-section at Buchanan Dam is shown in Figure 2.4S.4-2.

Mansfield Dam, at about 305 river miles upstream of STP 3 & 4, has a 2710 ft long, 266.4 ft high concrete gravity section occupying the main river channel, and a 4380 ft long earthen rockfill saddle section with a maximum height of about 150 ft on the left side (looking downstream) (Reference 2.4S.4-8). The total storage capacity is 3.13 MAF at the dam crest elevation of 750.1 ft MSL. With the installation of the 4-ft parapet wall in 1941, the storage capacity increased to 3.30 MAF. Following the FERC guidelines (Reference 2.4S.4-11), about half of the 2710 ft concrete gravity section was postulated to fail when overtopped by 3 ft, resulting in a 1360 ft wide vertical breached section from top to bottom. The time to complete the breach was also assumed to be 0.1 hour. The model cross-section for Mansfield Dam is shown in Figure 2.4S.4-3.

Table 2.4S.4-3 lists the dam breach characteristics used to model the failure of these two dams.

2.4S.4.2.1.3 Channel Geometry

The channel geometry in the HEC-RAS dam break model was adopted from the river cross-sectional data of Halff's flood damage evaluation study for the Lower Colorado River (Reference 2.4S.4-10 and discussed in Subsection 2.4S.4.3). The Halff model has a total model reach length of 474 river miles represented by 1048 cross-sections from Texas Highway 190 upstream of Buchanan Dam, to a section at 4600 ft (0.9 river miles) upstream of the Gulf Intracoastal Waterway just north of Matagorda Bay. The HEC-RAS dam break model developed for STP 3 & 4 has a shorter river reach of 414 miles starting from Buchanan Dam on the upstream end and was represented by a total of 793 model cross-sections. All bridge crossings specified in the Halff model were removed because they were assumed to be washed away during the dam break event. In addition, all ineffective flow areas as well as levees specified in the Halff model were also removed, when deemed appropriate. The locations of these cross-sections are shown in Figure 2.4S.4-4. The elevations of each of the cross-sections were referenced to the North America Vertical Datum 1988 (NAVD 88) in the Halff study. The HEC-RAS dam break model runs were also conducted in NAVD 88 datum. However, the flood level predictions were converted to MSL (or NGVD 29) for comparison with the STP plant grades.

Because the top-of-dam storage at Buchanan Dam was estimated to be 1.18 MAF, while the aggregated total top-of-dam storage of the 56 selected dams upstream of Buchanan Dam was estimated to be 6.87 MAF, it would not be possible for Buchanan Dam to accommodate the entire dam break flood volume from the breaching of these upstream dams. In order to properly account for the residual flows that could still arrive at and propagate downstream of Buchanan Dam after its failure, new model cross sections were introduced upstream of Buchanan Dam to extend the model reach by 36 miles to approximate the additional volume required to accommodate the combined dam break flood flow of 6.87 MAF from the dams upstream. The upstream reach extension consists of 37 rectangular cross sections 16,030-ft wide with a bottom elevation at 915.8 ft MSL. The cross-sectional width of 16,030 ft is similar to those of the three cross-sections behind Buchanan Dam in the Halff model (Reference 2.4S.4-10). The total flood volume in the model simulation would be over 8.0 MAF behind Buchanan Dam when it breaches at 3 ft above dam crest.

The primary objectives of the Halff study are for flood damage evaluations of the Lower Colorado River and therefore the model predictions were conducted for flood events up to the SPF. During extreme floods, inter-basin spillage could occur. Flood flow from the Colorado River could overspill into its neighboring sub-basins, such as Tres Palacios River to the west and San Bernard River and Peyton Creek to the east. In the flood of 1913, floodwaters from the Colorado River sub-basin overflowed into Caney Creek sub-basin to the east of the Colorado River near Wharton. With predictably higher flood discharges during the postulated dam failure scenario, the channel cross sections of the Halff study need to be extended beyond their limits to more accurately reflect the additional floodplain areas that would be inundated during the passage of the dam break flood waves. As HEC-RAS would automatically assume a vertical wall at the pre-set boundaries of the flood channel or floodplain, the extension could mitigate potentially unrealistic flood levels as a result of artificial limitation on the cross-sectional geometries imposed by the model setup. This can have a significant impact on the predicted flood peak in the lower reach of the river near the STP 3 & 4 site, where the drainage divides between sub-basins are relatively low in elevation.

A comparison was made between the simulated water levels from the initial dam break runs and the elevations of the drainage divides to determine the approximate location where inter-basin spillage would occur. It was found that inter-basin spillage could occur near Garwood. Therefore, about 1.9-mile extension was added to the Halff model cross sections on each side starting from near Garwood. The width of the extension on each side was gradually increased to about 9.5 miles near Wharton down the river. Because the topography is, in general, higher west of the Colorado River towards the Palacio River sub-basin, the cross-sectional extensions in the downstream reach shifted eastward towards the San Bernard River and the Peyton Creek sub-basins. Eventually, near the STP 3 & 4 site, the river cross-sections were extended towards the east for some 17 miles. Typical model cross-sections at four locations on the model river reach including the extended sections are shown in Figures 2.4S.4-5 to 2.4S.4-8.

The USGS 30-m National Elevation Dataset (NED) digital elevation model data used to establish the cross-sectional extensions was referenced to MSL (or NGVD 1929), while the Halff model was referenced to NAVD 88. As the difference between these two datum references for this reach of the Lower Colorado River is less than 0.3 ft, no corrections to the datum, except for 32 sections, were made to adjust the elevations of the extensions to NAVD 88 datum. The 32 sections with datum corrected were located between the STP site and the downstream boundary and were adopted from the PMF routing model described in Subsection 2.4S.3.

The locations and extents of the cross-sections used in the HEC-RAS dam break model are shown in Figure 2.4S.4-4.

2.4S.4.2.1.4 Manning's *n* Values Used in the HEC-RAS Model

The Manning's *n* values used in the Halff HEC-RAS model were calibrated with historical storms and measured flood levels using the values suggested in Table 2.4S.4-4 (Reference 2.4S.4-10) as initial estimates. The calibrated values are in the range of 0.025 to 0.046 for the river channel and 0.045 to 0.100 for the overbank areas, and they were used in the Halff study to model flood conditions up to the SPF. The extensions in the dam break model adopted the same Manning's *n* values assigned to the boundary limits of original cross-sections of the Halff model.

In a dam break event, there could be considerable amount of turbulence and entrainments of debris for many miles downstream of the breached section. In addition, a dam break flood, potentially with entrained debris, could overflow the river banks into the flood plains as well as inhabited areas, where the roughness could be considerably higher than those under severe flood conditions such as a SPF. To account for these conditions, the Manning's *n* values used by Halff in its HEC-RAS model were adjusted upward conservatively by a factor of 2.0 for 4 miles immediately downstream from the each of the failed dams, i.e., 4 miles downstream from Buchanan Dam and Mansfield Dam, respectively. For the rest of the model river reach, the

Manning's *n* values were assumed to be 1.2 times that used in the Halff study (Base Case). A sensitivity case was performed using the same Manning's *n* values as in the Halff study, except for a 4-mile distance downstream from Buchanan Dam as well as from Mansfield Dam where the Manning's *n* values were two times the values used in the Halff study (Sensitivity Case).

2.4S.4.2.1.5 Predicted Water Levels at STP 3 & 4 from Upstream Dam Failure Model

The HEC-RAS dam breach and unsteady flow routing model (Base Case) predicted that the peak water level at the STP site, without considering the wind wave effects, due to the domino-type failure of the upstream dams would be at El. 28.6 ft MSL or 28.4 ft (NAVD 88). The discharge at the time of the peak water level would be 1.87 x 106 cfs. For the Base Case, the flood wave would take about 65 hours to reach STP 3 & 4 after Mansfield Dam fails. This flood wave travel time would be about 58 hours for the Sensitivity Case. The predicted dam break flood and stage hydrographs for the two cases are presented in Figures 2.4S.4-9 and 2.4S.4-10. The simulated maximum dam break water surface profile from Buchanan Dam to the downstream boundary for the Base Case and Sensitivity Case are depicted in Figures 2.4S.4-11 and 2.4S.4-12, respectively.

2.4S.4.2.2 MCR Breach Analysis

The depth averaged two-dimensional (2-D) feature of the Delft3D-FLOW (Reference 2.4S.4-12) was used to evaluate the flooding potential due to the breaching of the MCR embankment. Delft3D-FLOW is a multi-dimensional hydrodynamic and transport numerical model which simulates non-steady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary fitted grid length. The model solves the Navier-Stokes equation for incompressible fluid using the shallow water and the Boussinesq assumptions. In addition, for 3D-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 equation 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 2D domain). Obstructions, such as buildings and embankments can be incorporated into the model.

For simulating flood levels from the breach of the MCR, the model domain was delineated in such a way that the entire MCR is included, together with the areas surrounding the power blocks of STP 1 & 2 and STP 3 & 4, the Essential Cooling Pond (ECP) of STP 1 & 2, and the Ultimate Heat Sink (UHS) of STP 3 & 4. The southern and eastern limits of the model domain align closely with the southern and eastern embankments of the MCR. The western and northern boundaries of the model were selected with the consideration that the maximum flood level would occur at the STP 3 & 4 power block before the flood waves reach these two downstream boundaries. No-flow boundary condition was applied to the four external boundaries of the model domain.

The model domain covers an area of approximately 6910 hectares (or 17,080 acres): 6990 m (or 4.3 miles) in the west-east direction and 9890 m (or about 6.1 miles) in the north-south direction. Table 2.4S.4-5 lists the coordinates of the four corners of model domain. The numerical grid for the model was generated with Delft3D-RGFGRID module: the horizontal grid size at the power block for STP 3 & 4 is 10 m by 10 m (or 32.8 ft by 32.8 ft), the grid size for the areas away from the power block is 20 m by 20 m (or 65.6 ft by 65.6 ft), and the grid size for transitional region is 10 m by 20 m (or 32.8 ft by 65.6 ft). Because the principal direction of the propagation of the flood waves is from the south to the north, the model was also oriented in the north-south direction. Figures 2.4S.4-13 and 2.4S.4-14 show the numerical grid of the MCR embankment breach model.

In addition to the safety related buildings and UHS of STP 3 & 4, the access road to the UHS was also represented in the Delft3D-FLOW model. Features of STP 1 & 2 represented in the model include the MCR embankments, ECP and the safety related buildings. All these features were modeled as “dry points” in which the flows perpendicular to the four faces of the grid cells, representing the buildings and the embankments, are blocked. Table 2.4S.4-6 depicts the buildings for which the “dry points” option was invoked. In addition, Figures 2.4S.4-15 and 2.4S.4-16 show the modeled and the physical locations of the building outlines, represented by green and blue lines, respectively.

2.4S.4.2.2.1 Assumptions in the MCR Breach Analysis

In the MCR breach analysis, the following assumptions were adopted:

(1) The failure and removal of the breached section in the MCR embankment would be instantaneous;

(2) All internal dikes within the MCR would also fail and be removed instantaneously, coincide with the breaching of the MCR embankment;

(3) The STP 1 & 2 Essential Cooling Pond (ECP) and the Ultimate Heat Sink (UHS) of STP 3 & 4 were modeled as structures with vertical walls (no flow-through conditions);

(4) The bottom elevation of the MCR was assumed to be uniform at El. 20 ft MSL and the initial reservoir water level would be at El. 50.74 ft MSL corresponding to a one half local PMP event (based on the local 72-hr PMP of 55.7 in. as stated in Subsection 2.4S.2) on top of the normal maximum MCR operating water level of El. 49 ft MSL. The reservoir storage volume at this MCR level (El. 50.74 ft MSL) is about 215,200 AF;

(5) The flow velocities in the MCR are zero before the instantaneous breach of the embankment;

(6) The Manning’s *n* value was selected to be 0.046;

(7) The density of water is 1000 kg/m (or 1.94 slug/ft) and the background horizontal eddy viscosity is 1.0 m/s (or 10.8 ft/s), which are the default values of Delft3D-FLOW. Because inertial forces dominate the dam break flow field, the effect of eddy viscosity would not be significant and has been verified in a sensitivity test.

2.4S.4.2.2.2 Bathymetry Elevations of the MCR Breach Model

The model bathymetry, also the elevation of the bottom boundary, was established using: (1) 2007 aerial topographic survey data of the STP 3 & 4 site; (2) USGS Digital Elevation Model (DEM) data of the area (Matagorda, Palacios NE, Wadsworth, and Blessing SE tiles); and (3) grading plan of STP 3 & 4 power block as shown in Figure 2.4S.4-17. For the model area outside the coverage of the aerial survey and the grading plan, the USGS DEM data was used and the interface between the data sets is indicated in Figure 2.4S.4-18. Bathymetric data was incorporated into the model with the Delft3D-QUICKIN module (Reference 2.4S.4-12). Figures 2.4S.4-19 and 2.4S.4-20 show the model representation of the bathymetry for the entire model, and for the power block area where the safety related structures are located. Bathymetric data is referenced to MSL and therefore any ground elevation above MSL would have a negative value. The power block is rectangular in plan of about 1718 ft (523.6 m) by 1286 ft (392.0 m). The grade elevation at the center of the power block is at 36.6 ft MSL and slopes to El. 32.0 ft MSL at the four corners.

The bottom elevations of the MCR vary approximately between El. 16.0 ft MSL at the southern end to El. 28.0 ft MSL at the northern end. These elevations correspond more or less to the natural ground topography before the building of the MCR. In the model, the entire MCR adopted conservatively a constant elevation of 20 ft MSL which is representative of the lowest lying area within the MCR.

2.4S.4.2.2.3 Boundary Conditions of the MCR 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 STP 3 & 4 safety related buildings due to a MCR breach would occur before the flood wave front reaches the two boundaries.

2.4S.4.2.2.4 Initial Conditions of the MCR Breach Model

The initial water level in the MCR was specified at El. 50.74 ft MSL corresponding to the local one half PMP (as discussed in Subsection 2.4S.2, the local 72-hr PMP is 55.7 in.). Outside of the MCR, three different initial downstream flood levels: El. 32.0 ft, El. 34.0 ft and dry condition, were evaluated as part of a sensitivity test. The maximum flood level at the safety related facilities of STP 3 & 4 were found to be independent of the initial flood depths within the plant site.

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

2.4S.4.2.2.5 Selection of the MCR Breach Model Parameters

The surface roughness in the model was represented by Manning's *n* value. Based on the UFSAR of STP 1 & 2, Reference 2.4S.4-5, Manning's *n* was specified as 0.046 uniformly in the two principal directions (east-west and north-south) throughout the model domain. This relatively high Manning's *n* was used to account for the smaller buildings and structures between the MCR and the power blocks of STP 1 & 2 and STP 3 & 4 that were not specifically included in the model.

The simulations were run at a model time step of 0.01 minutes (0.6 seconds), which was selected based on a verification effort to demonstrate the time-step independence of the model results.

2.4S.4.2.2.6 Flood Levels from the MCR Breach

Similar to the approach used in the MCR breach simulation detailed in UFSAR of STP 1 & 2, multiple embankment breach widths (also referred to as breach lengths) were investigated with the Delft3D model. The breached widths simulated vary from 190 m (or 623 ft) to 1690 m (or 5545 ft), with the centerline of the breached section aligned with the centerline of the STP 3 & 4 reactor buildings. The resulting maximum flood levels at the safety buildings in the STP 3 & 4 power block for the various simulated breached widths are presented in Table 2.4S.4-7, which indicates that a maximum flood level of El. 47.6 ft MSL at STP 3 & 4 would occur at a breached width of about 1450 m (or 4757 ft). This maximum flood level would occur at the southern face of the STP 4 Reactor Building. However, the southern faces of STP 3 & 4 Radwaste Buildings also experience high flood levels. For design purpose, all safety related buildings including the UHS for STP 3 & 4 are designed against the maximum flood level of 47.6 ft MSL.

Figure 2.4S.4-21 details the time history of the simulated flood level at the southern face of the STP 4 Reactor Building. As indicated in the figure, the flood wave arrives at the building in about 2 minutes after the embankment breaches, and a quasi-steady state flow regime is sustained for about 13 minutes (between 7 and 20 minutes after the embankment breach). Thereafter, the flood level drops because of the receding storage volume and water level in the MCR.

Coincidental wind set-up and wave run-up were not added to the highly conservative MCR breach flooding level because this flooding has a short time scale and would not sustain for a period long enough for any considerable wind-wave action. Further, the buildings and facilities in the vicinity of the safety-related structures of STP 3 & 4 would have limited the fetch to a small distance such that the generation of effective wind waves is considered unlikely.

The static and dynamic effects of the MCR's northern embankment breach on the plant structures are discussed in Section 3.4.

2.4S.4.3 Water Level at the STP 3 & 4 Site

Analyses of the dam failures on the Lower Colorado River and the failure of the MCR northern embankment showed that the critical flood level of the safety related structures is controlled by the MCR embankment failure. The design basis flood level for the safety related facilities of STP 3 & 4 is therefore 47.6 ft MSL as discussed below.

2.4S.4.3.1 Water Level at the STP 3 & 4 Site from the Failures of Upstream Dams

In accordance with the guidelines in ANSI/ANS-2.8, Reference 2.4S.4-7, the maximum dam breach flood level at the plant site needs to consider the wind setup and wave runup effect from the coincidental occurrence of a 2-year design wind event. The 2-year fastest mile wind speed at the site is 50 mph based on Reference 2.4S.4-7. The methodology given by the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, was adopted to estimate the wave height and wave run-up at STP 3 & 4 power block. The procedures outlined in CEM use the wind speed, wind duration, water depth, and over-water fetch distance, and the run-up surface characteristics as input. As discussed in UFSAR for STP 1 & 2 (Reference 2.4S.4-5), accurate estimates of the fetch length for this flooding scenario could not be made. Based on the topographic variations and any man-made features that would limit wind effects, however, two critical fetches were identified as shown in Figure 2.4S.4-22; one in an easterly direction towards a low lying ridge and the other along the Colorado River in a northeasterly direction. The fetch in the easterly direction was estimated to be about 15.5 miles with a maximum water depth varying from 1 to 23 ft at the peak of the dam break flood. The fetch along the northeasterly direction was estimated to be about 17.6 miles, with a maximum water depth varying from 1 to 9 ft at the flood peak.

The maximum wind set-up for the critical fetch lines was estimated using a method suggested in Reference 2.4S.4-14, and was found to be about 3.9 ft. Adding to the maximum water level of El. 28.6 ft MSL, estimated by the HEC-RAS dam break model for the STP site, the water level from the dam failure flooding scenario would therefore be at El. 32.5 ft MSL. With the surrounding site grade around the power block and UHS at a nominal elevation of 28.0 ft MSL, the water depth approaching at the STP power block and UHS would be about 4.5 ft. At this shallow depth, a breaking wave condition would prevail and a breaking wave index of 0.78 was used in estimating the break wave height. The breaking wave setup is typically small and is assumed to have a negligible impact on the flood level.

The maximum wave run-up was estimated using the breaking wave height of 3.5 ft and a maximum wave period equal to 1.2 times of the significant wave period which was estimated to be 3.7 seconds. Conservatively assuming that the run-up surface is smooth, impermeable and at a slope of 2H:1V, the wave run-up was estimated to be about 9.4 ft.

The maximum flood level at STP 3 & 4 power block as a result of the probable worst case dam failure scenario coincidental with a 2-year design wind of 50 mph was estimated to be at El. 41.9 ft MSL. Table 2.4S.4-8 presents the water levels due to dam break, wind set-up and wave run-up at STP 3 & 4 for the two critical fetches.

Because the STP is about 300 miles from Mansfield Dam, any dynamic effects of the dam break waves would have been attenuated along this distance. Therefore, the dynamic effects of the dam break flood waves are not the controlling design criterion of the safety related facilities.

2.4S.4.3.2 Water Level at the STP 3 & 4 Site from Breaching of MCR Embankment

The maximum water level at STP 3 & 4 is governed by the postulated breaching of the MCR's northern embankment. The maximum water level at the power block and UHS of STP 3 & 4 due to the breaching of the MCR's northern embankment is at El. 47.6 ft MSL. Because the maximum water level is higher than the nominal plant grade of 34.0 ft MSL as well as the entrance level slab elevation of 35.0 ft MSL for the STP 3 & 4 safety related facilities, all safety related facilities are designed to be water tight at or below elevation 47.6 ft MSL. All ventilation openings of safety buildings are located at 47.6 ft MSL or above. Flood protection design is discussed in Subsection 2.4S.10 and Section 3.4.

2.4S.4.3.3 Sedimentation and Erosion

During an upstream dam failure event, because the plant site is located in the floodplains of the Colorado River, the flow velocities are expected to be relatively small compared to that in the main channel. In addition, the flow depths on the floodplain are shallower to effect any significant erosion that would impact the safety of the plant. Although some sedimentation may occur near the plant site, the safety related structures and functions would not be affected by siltation because they are located at higher grades than the surrounding area.

The erosion concern during a MCR embankment breach event is discussed in Subsection 2.4S.10.

2.4S.4.4 References

2.4S.4-1 "Texas Administrative Code – Title 30, Part 1, Chapter 299," Office of the Secretary of State of Texas, provisions adopted to be effective May 13, 1986 (11 TexReg 1978).

2.4S.4-2 "Phase II – Dam Safety Evaluation Project, Task Order B – Reconnaissance investigation, Interim Report," Volume I, Freese and Nichols, Inc., August 1992.

2.4S.4-3 "Celebration marks completion of 10-year LCRA dam project to improve public safety," Press Release by LCRA dated January 12, 2005; available at http:/www.lcra.org/newsstory/2005/dam_upgrade_project.html, accessed on August 31, 2007.

2.4S.4-4 "Disaster Ready Austin: Building a Safe, Secure and Sustainable Community," City of Austin Hazard Mitigation Action Plan, 2003 – 2008, prepared by LCRA and H2O, Inc., revised on August 7, 2003.

2.4S.4-5 STPEGS Updated Final Safety Analysis Report, Units 1 & 2, Revision 13.

2.4S.4-6 "Water for Texas – 2007," Volumes I, II, and III, Texas Water Development Board, January 2007.

2.4S.4-7 Determining Design Basis Flooding at Power Reactor Sites," La Grange Park, Illinois, ANSI/ANS-2.8-1992, American Nuclear Society, July 1992. (Historical Technical Reference)

2.4S.4-8 "Engineering Data on Dams and Reservoirs in Texas," Part III, Report 126, Texas Water Development Board, February 1971.

2.4S.4-9 "HEC-RAS, River Analysis System, Version 3.1.3," U.S. Army Corps of Engineers, Hydrologic Engineering Center, May 2005.

2.4S.4-10 "Flood Damage Evaluation Project," Chapter 1-6, Volume II-C, Volume II-B, Halff Associates, Inc., July 2002.

2.4S.4-11 "Industries Regulations, Guidelines and Manual – Engineering Guidelines for the Evaluation of Hydropower Projects," Federal Energy Regulatory Commission, April 1991.

2.4S.4-12 "WL|Delft, 2005: Delft3D-FLOW, Simulation of multi-dimensional hydrodynamic flows and transport phenomena, including sediments," User Manual, Delft, The Netherlands.

2.4S.4-13 "Coastal Engineering Manual," U.S. Army Corps of Engineers, June 2006.

2.4S.4-14 "Advanced Series on Ocean Engineering, Volume 16, Introduction to Coastal Engineering and Management," J. William Kamphuis, 2000.

2.4S.4-15 "Colorado River – Flood Guide," Lower Colorado River Authority, Texas, January 2003.

Table 2.4S.4-1 Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity

No.	Dam Name	County	Height of Dam (ft)	Length of Dam (ft)	Top of Dam Elevation (ft MSL)	Maximum Capacity (AF at top of dam)	Dam Type	Date of Completion
01	Mansfield Dam	Travis	266.4	7,089	750.1 (754.1 ft: top of parapet)	3,300,000 [4]	Concrete Gravity Earth and Rockfill	1942
02	Simon Freese Dam [5]	Coleman	148	15,950	1584	1,470,000 [4]	Earth and Rock Fill Embankment	1990
03	Twin Buttes Dam [5]	Tom Green	134	42,460	1991	1,294,000 [3]	Earthfill	1963
04	Buchanan Dam	Burnet	145.5	10,987	1025.35	1,180,000 [1]	Multiple Concrete Arch, Gated and Gravity sections	1937
05	Robert Lee Dam [5]	Coke	140	21,500	1928	1,074,000 [3]	Earthfill	1969
06	O C Fisher Dam [5]	Tom Green	128	40,885	1964	815,000 [2]	Earthfill	1952
07	Brownwood Dam [5]	Brown	120	1,580	1449.5	448,2000 [1]	Earthfill	1933
08	Lake J B Thomas Dam [5]	Scurry	105	14,500	2280	431,000 [2]	Earthfill	1952
09	Alvin Wirtz Dam	Burnet	118.29	5,491	835.25	226,000 [4]	Concrete and Earthfill	1951
10	Brady Dam [5]	McCulloch	104	8,400	1783	213,000 [3]	Earthfill	1963
11	Natural Dam [1] [5]	Howard	47	[6]	[6]	207,265	Earth	1989
12	Tom Miller Dam	Travis	85	1,590	519	115,404 [1]	Concrete Gravity	1939
13	Coleman Dam [5]	Coleman	90	3,200	1740	108,000 [3]	Earthfill	1966
14	Champion Creek Dam [5]	Mitchell	114	6,800	2109	103,600 [3]	Earthfill	1959
15	Cedar Creek Dam	Fayette	96	8,000	401	101,000 [4]	Earthfill	1977
16	Oak Creek Dam [5]	Coke	95	3,800	2104	83,800 [3]	Earthfill	1952
17	Colorado City Dam [5]	Mitchell	85	4,800	2090	78,400 [4]	Earthfill	1949
18	Hords Creek Dam [5]	Coleman	91	6,800	1939	66,300 [3]	Earthfill	1948
19	Roy Inks Dam	Burnet	96.5	1,547.5	922	63,500 [1]	Concrete Gravity	1938

Table 2.4S.4-1 Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity (Continued)

No.	Dam Name	County	Height of Dam (ft)	Length of Dam (ft)	Top of Dam Elevation (ft MSL)	Maximum Capacity (AF at top of dam)	Dam Type	Date of Completion
20	Mitchell County Dam [1] [5]	Mitchell	70	[6]	[6]	50,241	Earth	1991
21	Decker Creek Dam	Travis	83	6,390	563	45,300 [2]	Earthfill	1967
22	Nasworthy Dam [5]	Tom Green	50	5,480	1883.5	43,300 [4]	Earthfill	1930
23	Ballinger Municipal Lake Dam [1] [5]	Runnels	76	6,200	1,694	34,353	Earth	1985
24	Elm Creek Dam [1] [5]	Runnels	57	5,640	1,810	33,500	Earth	1983
25	Bastrop Dam	Bastrop	85	4,000	458	24,200 [1]	Earthfill	1964
26	Sulphur Springs Draw Dam [1] [5]	Travis	33	[6]	[6]	20,692	Earth	1993
27	Upper Pecan Bayou WS SCS Site 7 Dam [5]	Callahan	63	3,950	1888.9	20,000 [3]	Earthfill	1970
28	Brady Creek WS SCS Site 17 Dam [1] [5]	Mcculloch	50	4,208	[6]	13,511	Earth	1962
29	Brady Creek WS SCS Site 28 Dam [1] [5]	Concho	42	6,459	[6]	13,042	Earth	1957
30	Brady Creek WS SCS Site 31 Dam [1] [5]	Concho	50	5,910	[6]	11,155	Earth	1958
31	Old Lake Winters City Dam [1] [5]	Runnels	37	3,090	1800.2	10,032	Earth	1945
32	Eagle Lake Dam [2]	Colorado	Varies 6 ft +/-	5,300	Not known	9,600 at EL 170 ft, msl	Earthfill	1990
33	Brady Creek WS SCS Site 20 Dam [1] [5]	Concho	43	4,010	[6]	9,494	Earth	1959
34	Northwest Laterals WS SCS Site 5A Dam [1] [5]	Coleman	57	2,631	[6]	9,416	Earth	1971

Table 2.4S.4-1 Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity (Continued)

No.	Dam Name	County	Height of Dam (ft)	Length of Dam (ft)	Top of Dam Elevation (ft MSL)	Maximum Capacity (AF at top of dam)	Dam Type	Date of Completion
35	Max Starcke Dam	Burnet	98.8	860	766 [1] 738 [7]	8,760 [1]	Concrete with Roof-weir Gated	1951
36	Jim Ned Creek WS SCS Site 25 Dam [1] [5]	Coleman	44	2,400	[6]	8,368	Earth	1963
37	Jim Ned Creek WS SCS Site 12E1 Dam [1] [5]	Coleman	64	2,000	[6]	8,271	Earth	1965
38	Ballinger City Lake Dam [1] [5]	Runnels	30	4,400	1704.6	8,215	Earth	1947
39	Elm Creek WS_NRCS Site 3 Rev. [1] [5]	Runnels	39	[6]	[6]	8,165	Earth	2004
40	Clear Creek WS SCS Site 6 Dam [1] [5]	Brown	50	2,101	1461	8,083	Earth	1958
41	Jim Ned Creek WS SCS Site 21 Dam [1] [5]	Coleman	92	1,915	[6]	7,930	Earth	1963
42	Clear Creek WS SCS Site 4 Dam [1] [5]	Brown	45	2,300	1508.6	7,891	Earth	1958
43	Upper Pecan Bayou WS SCS Site 2 Dam [1] [5]	Callahan	69	2,025	1948.8	7,833	Earth	1967
44	Brady Creek WS SCS Site 14 Dam [1] [5]	Mcculloch	43	4,091	[6]	7,732	Earth	1956
45	Home Creek WS SCS Site 13 Dam [1] [5]	Coleman	45	2,410	[6]	7,679	Earth	1974
46	Valley Creek WS SCS Site 1 Dam [1] [5]	Nolan	52	5,100	2121.8	7,600	Earth	1968
47	Upper Pecan Bayou WS SCS Site 24 Dam [1] [5]	Coleman	50	1,800	1606.4	7,394	Earth	1972

Table 2.4S.4-1 Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity (Continued)

No.	Dam Name	County	Height of Dam (ft)	Length of Dam (ft)	Top of Dam Elevation (ft MSL)	Maximum Capacity (AF at top of dam)	Dam Type	Date of Completion
48	Brownwood Laterals WS SCS Site 3 Dam [1] [5]	Brown	83	1,930	1473.9	7,377	Earth	1973
49	Northwest Laterals WS SCS Site 1 Dam [1] [5]	Runnels	50	2,520	[6]	7,181	Earth	1964
50	Brady Creek WS SCS Site 32 Dam [1] [5]	Concho	32	8,075	[6]	7,053	Earth	1959
51	Longhorn Dam [1]	Travis	65	1,240	464	6,850	Earth, Gravity	1960
52	Jim Ned Creek WS SCS Site 23 Dam [1] [5]	Coleman	62	1,980	[6]	6,754	Earth	1962
53	Elm Creek WS NRCS Site 7 [1] [5]	Runnels	39.5	[6]	[6]	6,500	Earth	1998
54	Home Creek WS SCS Site 7A Dam [1] [5]	Coleman	48	3,396	[6]	6,367	Earth	1970
55	Jim Ned Creek WS SCS Site 12 Dam [1] [5]	Coleman	84	1,900	[6]	6,334	Earth	1963
56	Mukewater Creek WS SCS Site 10A Dam [1] [5]	Coleman	35	3,190	1485.7	6,130	Earth	1965
57	Elm Creek Lake Dam [1] [5]	Runnels	23	450	1635	6,018	Earth	1930
58	Clear Creek WS SCS Site 3 Dam [1] [5]	Brown	55	1,950	1451.5	5,988	Earth	1960
59	Se Laterals WS SCS Site 7 Dam [1] [5]	San Saba	43	2,225	[6]	5,899	Earth	1968
60	Brady Creek WS SCS Site 21 Dam [1] [5]	Concho	30	3,543	[6]	5,742	Earth	1958
61	Upper Pecan Bayou WS SCS Site 12 Dam [1] [5]	Callahan	65	1,400	1759.3	5,707	Earth	1967

Table 2.4S.4-1 Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity (Continued)

No.	Dam Name	County	Height of Dam (ft)	Length of Dam (ft)	Top of Dam Elevation (ft MSL)	Maximum Capacity (AF at top of dam)	Dam Type	Date of Completion
62	Moss Creek Lake Dam [1] [5]	Howard	67	2,450	2341.6	5,700	Earth	1939
63	Cummins Creek WS SCS Site 1 Dam [1]	Lee	25	4,050	450.9	5,627	Earth	1958
64	Brady Creek WS SCS Site 36 Dam [1] [5]	Concho	33	1,973	[6]	5,352	Earth	1955
65	Northwest Laterals WS SCS Site 2 Dam [1] [5]	Coleman	52	2,082	[6]	5,297	Earth	1964
66	Jim Ned Creek WS SCS Site 26A Dam [1] [5]	Coleman	46	4,000	[6]	5,280	Earth	1966
67	Jim Ned Creek WS SCS Site 19 Dam [1] [5]	Taylor	28	2,985	[6]	5,218	Earth	1960
68	Clear Creek WS SCS Site 1 Dam [1] [5]	Brown	40	1,542	1397.6	5,128	Earth	1960

[1] Data provided by TCEQ
[2] Data provided by TWDB: data was directly listed in Reference 2.4S.4-8
[3] Data provided by TWDB: data were extrapolated based on the storage-stage curves in Reference 2.4S.4-8
[4] Data provided by TWDB: data were extrapolated based on the storage-stage area data
[5] Dams located upstream of Buchanan Dam
[6] No information was given by TCEQ
[7] Data from LCRA in Reference 2.4S.4-15

Table 2.4S.4-2 500-year and SPF Inflow Peak Discharges at Selected Locations along the Colorado River (in cfs)

Flood Event	Buchanan	Mansfield	Tom Miller	Bastrop	Garwood	Wharton	Bay City
500-year	382,400	499,700	366,900	321,900	256,700	204,700	187,900
SPF	484,800	737,000	402,500	359,900	285,500	237,800	214,200

Source: Reference 2.4S.4-10

Table 2.4S.4-3 Breach Parameters for Buchanan and Mansfield Dams

Breach Parameters	Buchanan Dam	Mansfield Dam
Average Width of Breach (ft)	1470	1360
Breach Bottom Elevation (ft, MSL)	879.8	484
Breach Top Elevation (ft, MSL)	1,028.4	757
Side Slope of Breach	0	0
Breach Time to Failure (hrs)	0.1	0.1

Table 2.4S.4-4 Initial Estimation of Manning's Roughness Coefficient

n Values Assigned to the USGS NLCD Dataset		
USGS Classification Grid-Code	Description	*n* Value
11	Open water	0.03
21	Low intensity residential	0.07
22	High intensity residential	0.09
23	Commercial/industrial/transportation	0.10
31	Bare rock/sand/clay	0.04
32	Quarries/strip mines/gravel pits	0.035
41	Deciduous forest	0.095
42	Evergreen forest	0.085
51	Shrubland	0.08
71	Grasslands/herbaceous	0.04
81	Pasture/hay	0.045
82	Row crops	0.05
83	Small grains	0.055
85	Urban/recreation grasses	0.03
91	Woody wetlands	0.10
92	Emergent herbaceous wetlands	0.085

Source: Reference 2.4S.4-10

Table 2.4S.4-5 Coordinates of Model Domain Corner Points

Model Corners	Easting (m/ft)	Northing (m/ft)
Southwest	10,000.0/32,808.4	12,900.0/42,322.8
Southeast	16,990.0/55,741.5	12,900.0/42,322.8
Northwest	10,000.0/32,808.4	22,790.0/74,770.3
Northeast	16,990.0/55,741.5	22,790.0/74,770.3

Table 2.4S.4-6 List of Buildings Included in the MCR Breach Model

STP 3 & 4	STP 1 & 2
Reactor Building (No. 1)	Reactor Containment Building
Turbine Building (No. 2)	Mechanical-Electrical Auxiliaries Building
Control Building (No. 3)	Fuel-Handling Building
Radwaste Building (No. 4)	Diesel-Generator Building
Service Building (No. 5)	Turbine Building (TGB)
Hot Machine Shop (No. 6)	Isolation Valve Cubicle (IVC)
Passageway to Hot Machine Shop	

Table 2.4S.4-7 Variation of Maximum Flood Level with MCR Breach Width

Breach Width		Maximum Water Level (MSL)	
m	ft	m	ft
1,690	5,545	14.47	47.5
1,450	4,757	14.50	47.6
1,210	3,970	14.46	47.4
970	3,182	14.34	47.0
730	2,395	14.06	46.1
610	2,001	13.84	45.4
490	1,608	13.56	44.5
310	1,017	12.87	42.2
250	820	12.54	41.1
190	623	12.17	39.9
120	394	11.61	38.1
60	197	10.97	36.0

Table 2.4S.4-8 Estimated Water Levels due to Dam Break, Wind Setup, and Wave Run-up

	Dam Break Water Level (ft MSL)	Wind Setup (ft)	Wave Run-up (ft)	Water Level at STP Site (ft MSL)
Fetch A (I)	28.6	3.9	9.4	41.9
Fetch A (II)	27.8	4.2	8.4	40.4
Fetch B (I)	28.6	3.9	9.3	41.8
Fetch B (II)	27.8	4.0	7.9	39.7

Note: (I) - Base Case; (II) - Sensitivity Case

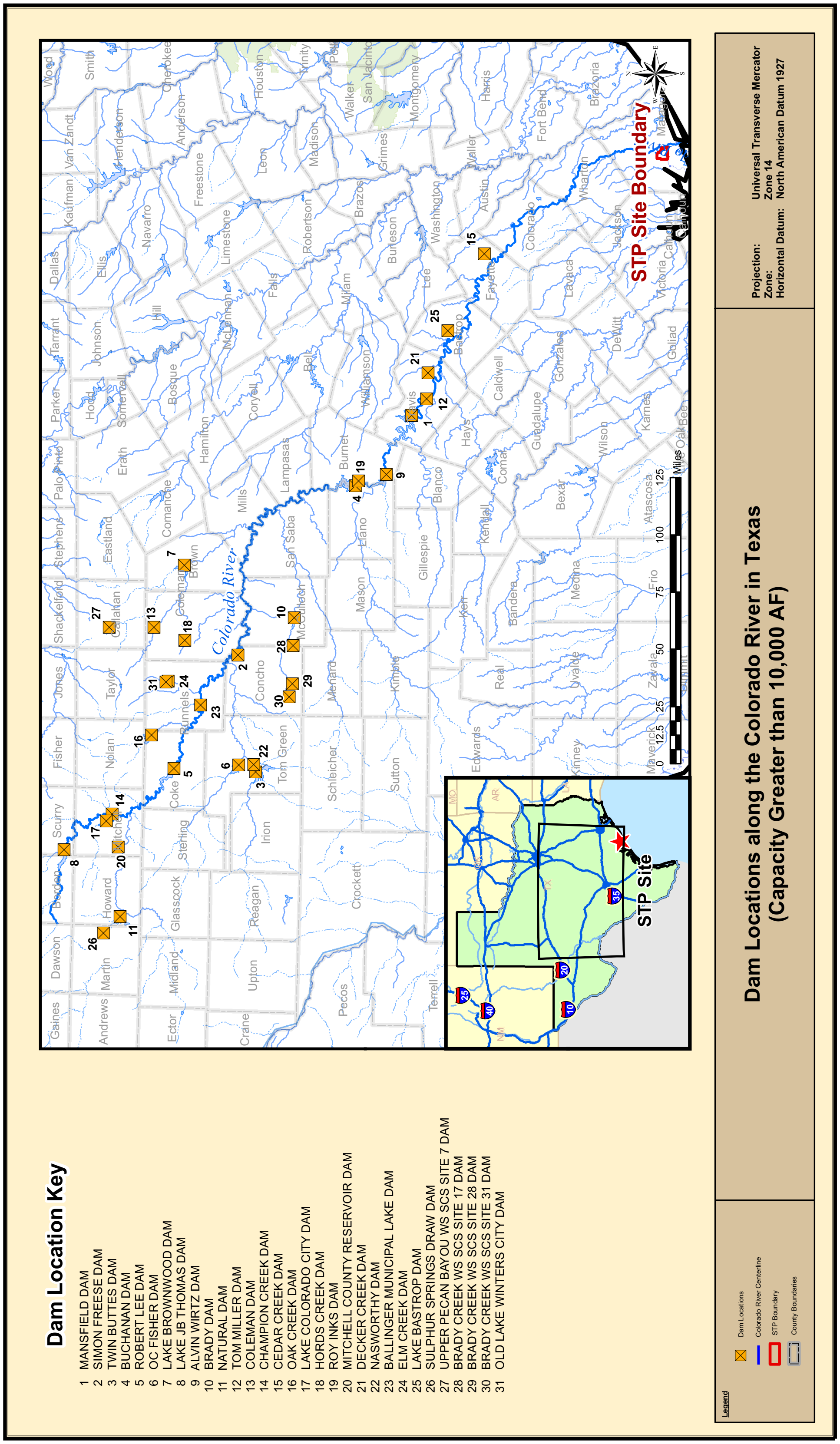


Figure 2.4S.4-1a Locations of Dams with Storage Capacity Over 10,000 AF in the Colorado River Basin Upstream of the STP 3 & 4 Site

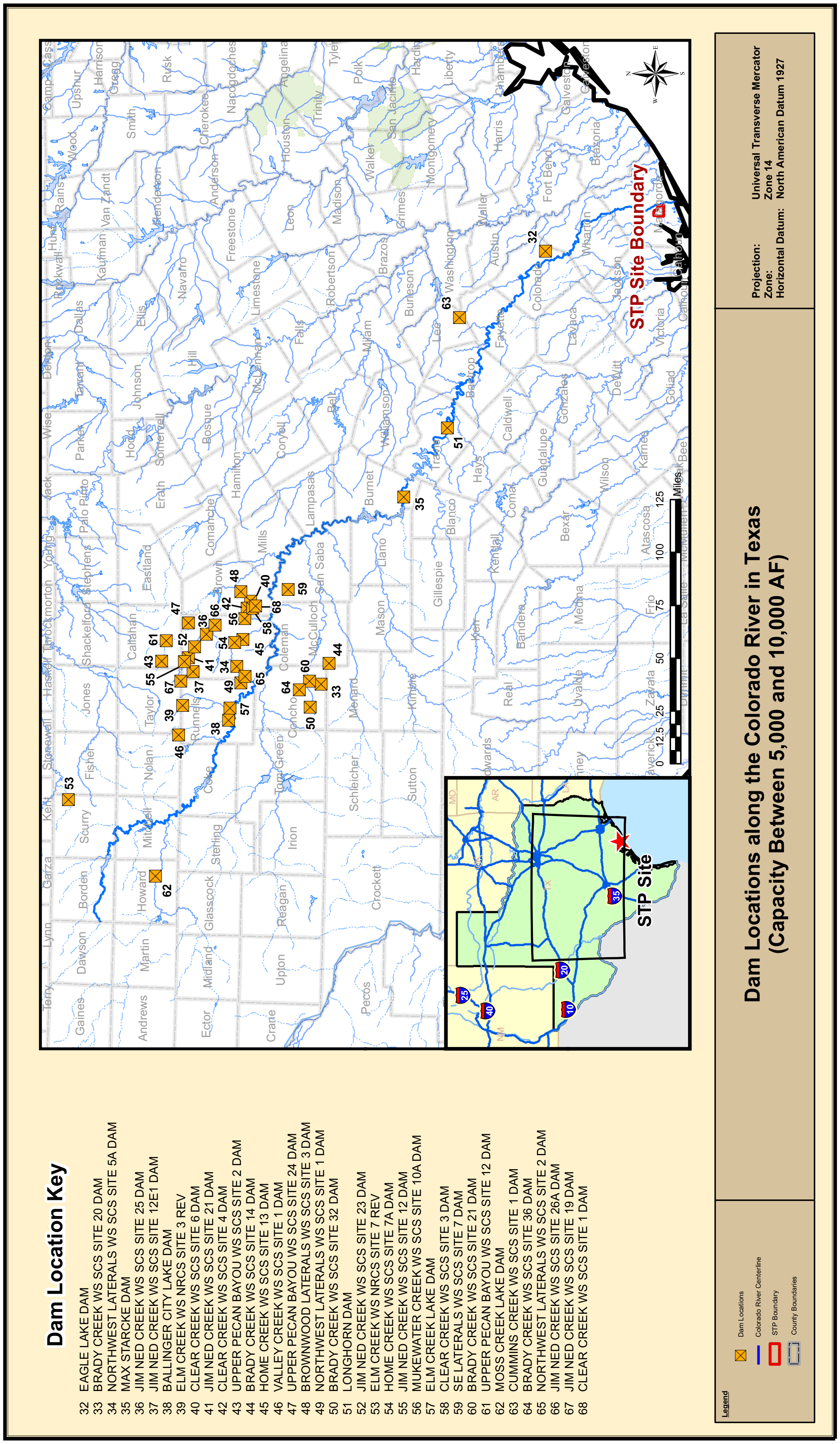


Figure 2.4S.4-1b Locations of Dams with Storage Capacity of 5,000 AF to 10,000 AF in the Colorado River Basin Upstream of the STP 3 & 4 Site

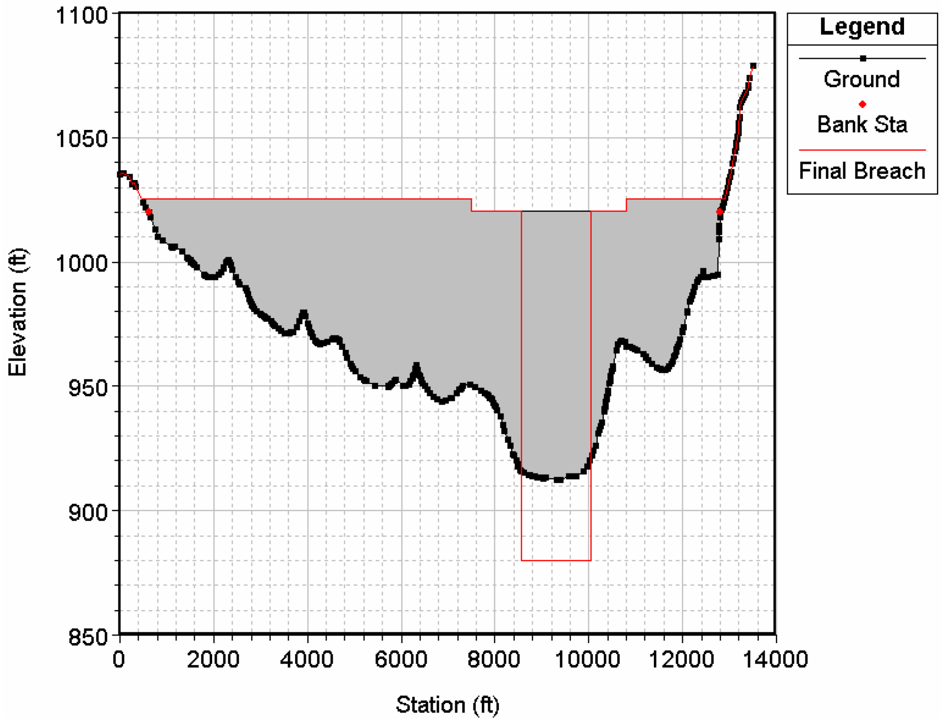


Figure 2.4S.4-2 Model Cross Section at Buchanan Dam

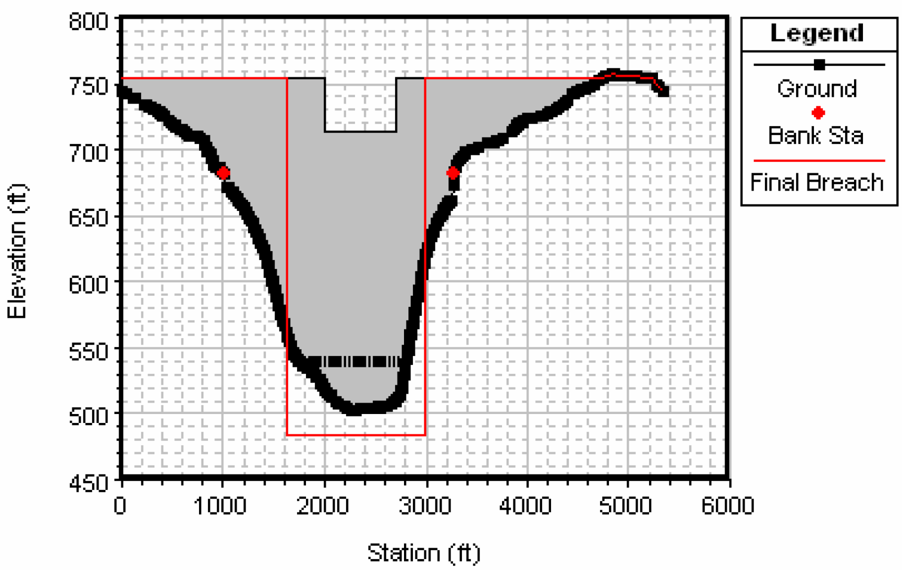


Figure 2.4S.4-3 Model Cross Section at Mansfield Dam

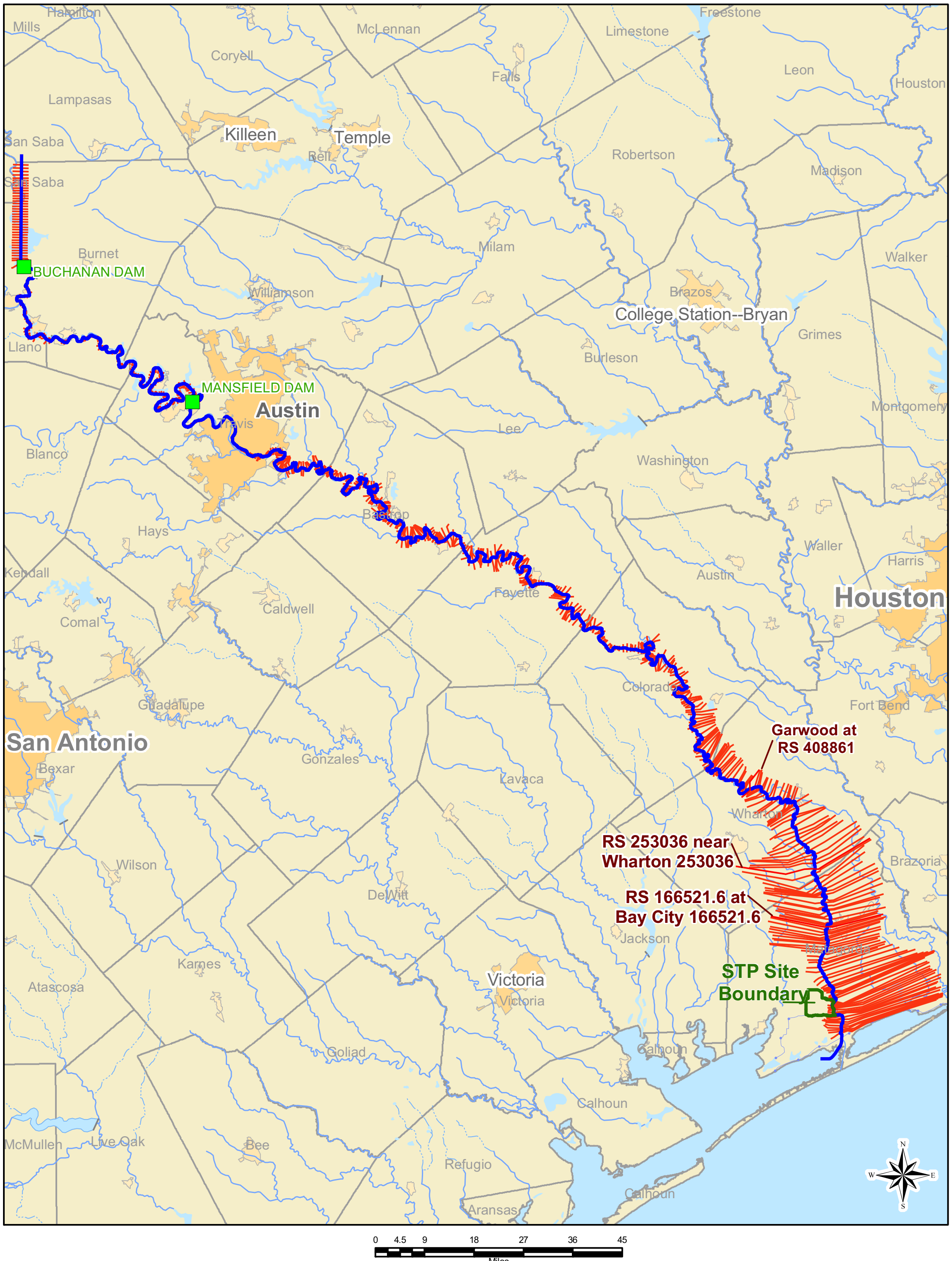


Figure 2.4S.4-4 Locations of Model Cross Sections in the Dam Break Analysis

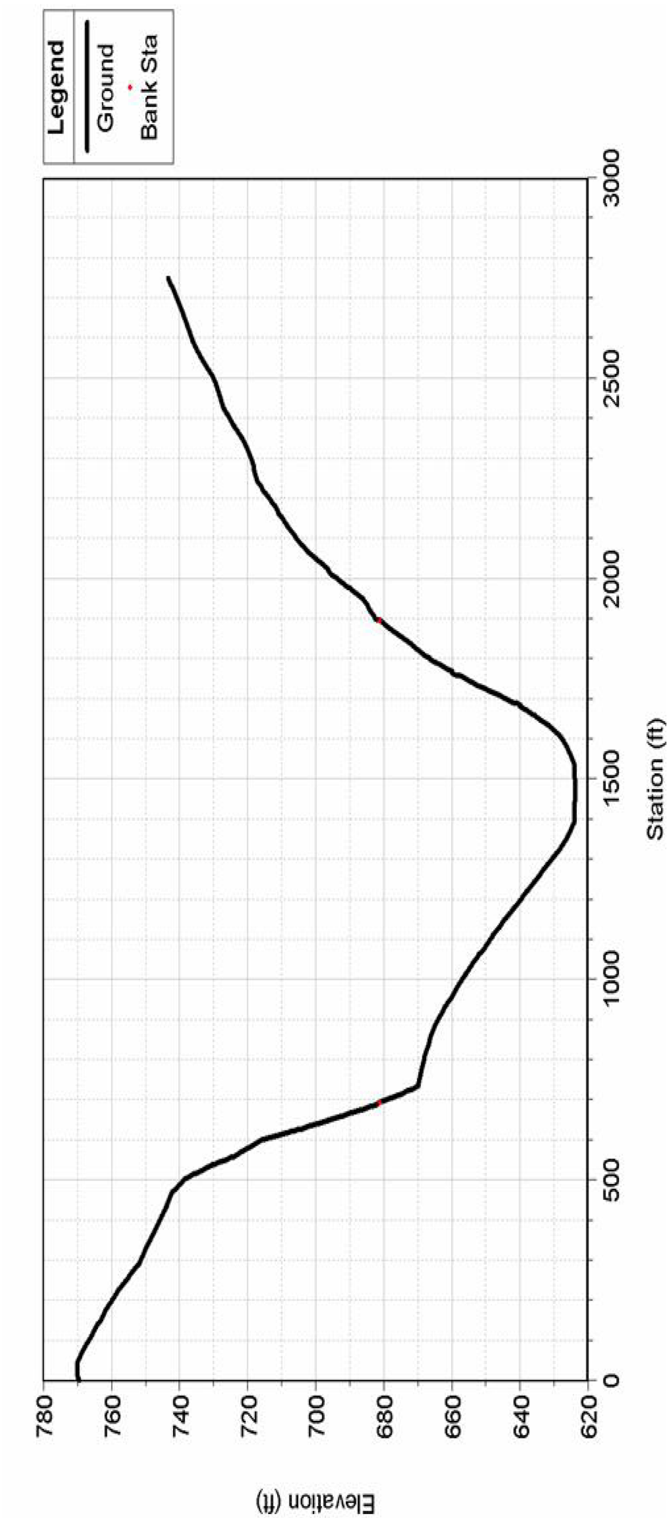


Figure 2.4S.4-5 Model River Cross Section at About 365 River Miles Upstream of the GIWW

Note: Between Buchanan and Mansfield Dams and about 49.6 River Miles Upstream of Mansfield Dam.

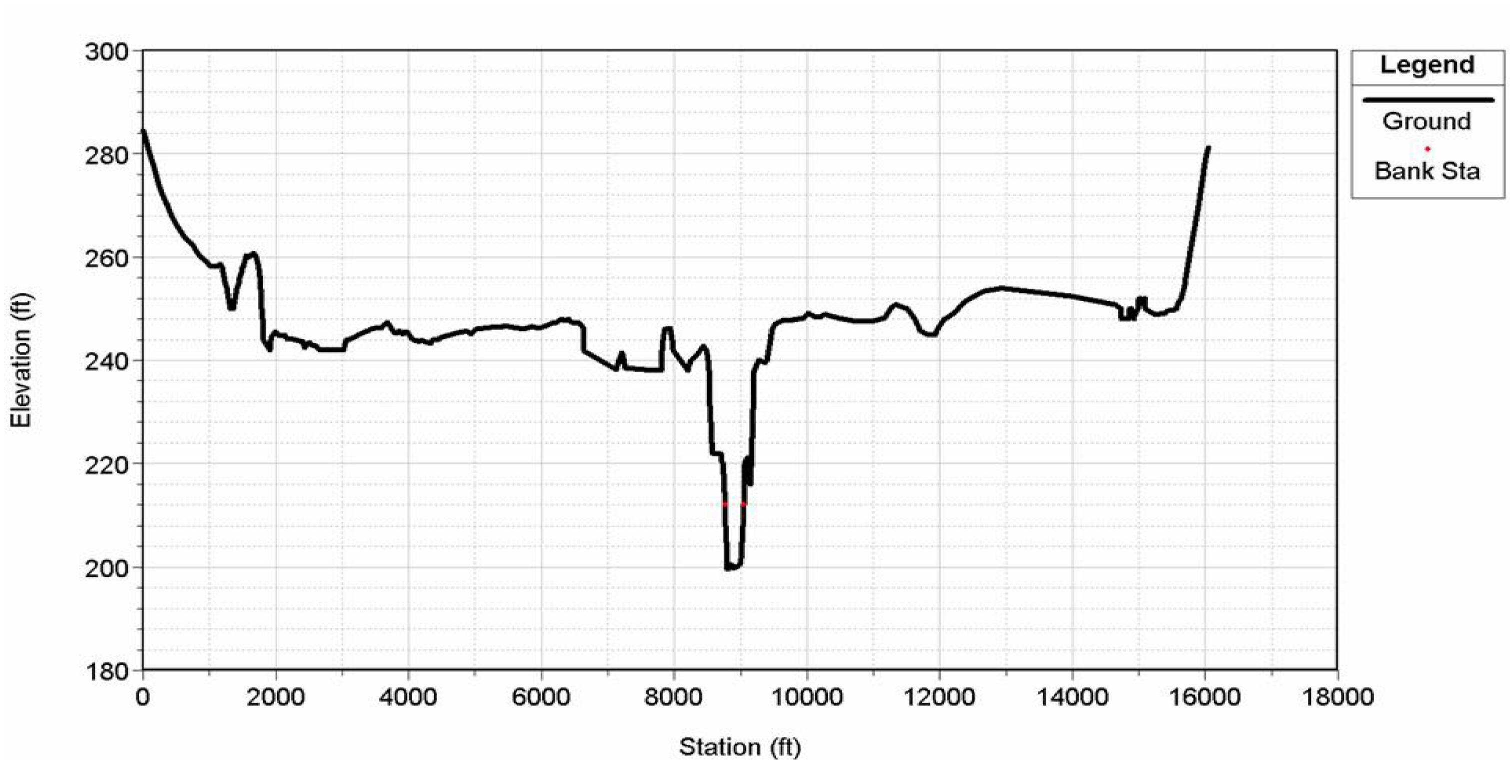


Figure 2.4S.4-6 Model River Cross Section at About 163.5 River Miles Upstream of the GIWW

Note: Downstream of Mansfield Dam and about 153 miles Upstream of STP 3 & 4 Site.

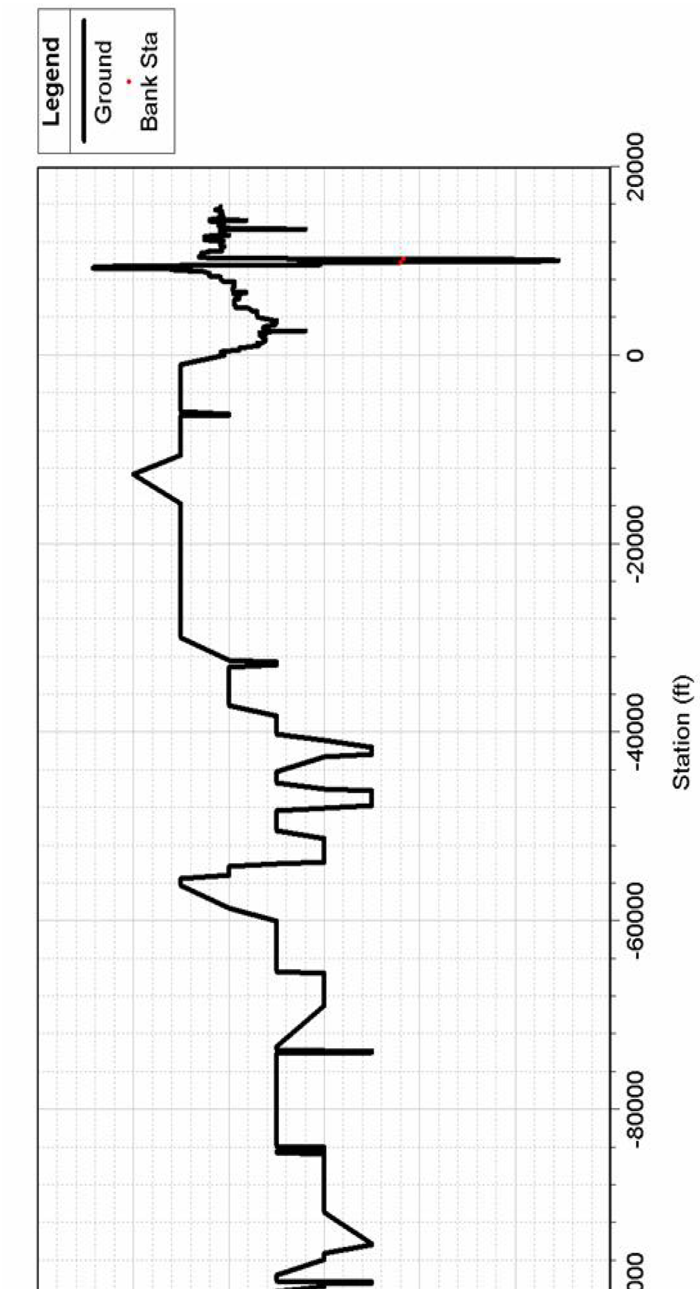


Figure 2.4S.4-7 Model River Cross Section at About 10.5 River Miles Upstream of the GIWW

Note: Near the STP site.

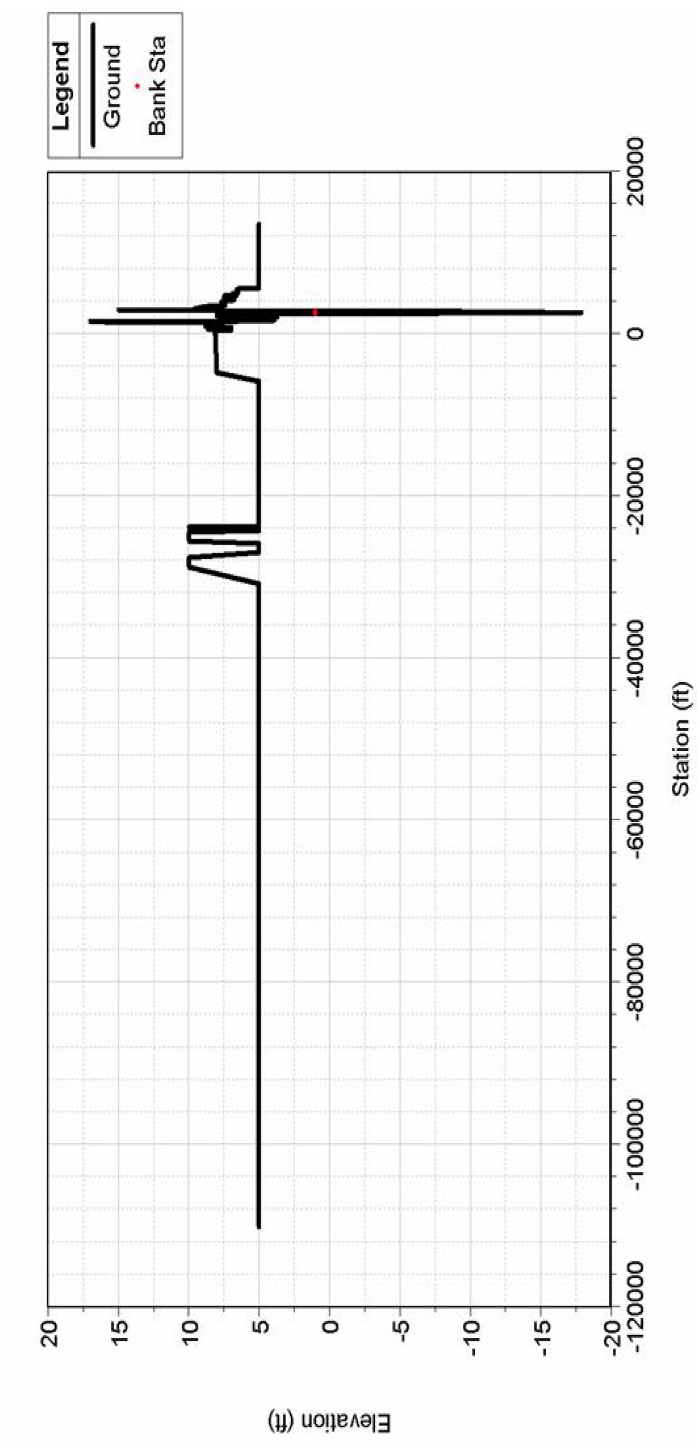


Figure 2.4S.4-8 Model River Cross Section at Downstream Model Boundary at about 0.9 River Miles Upstream of the GIWW

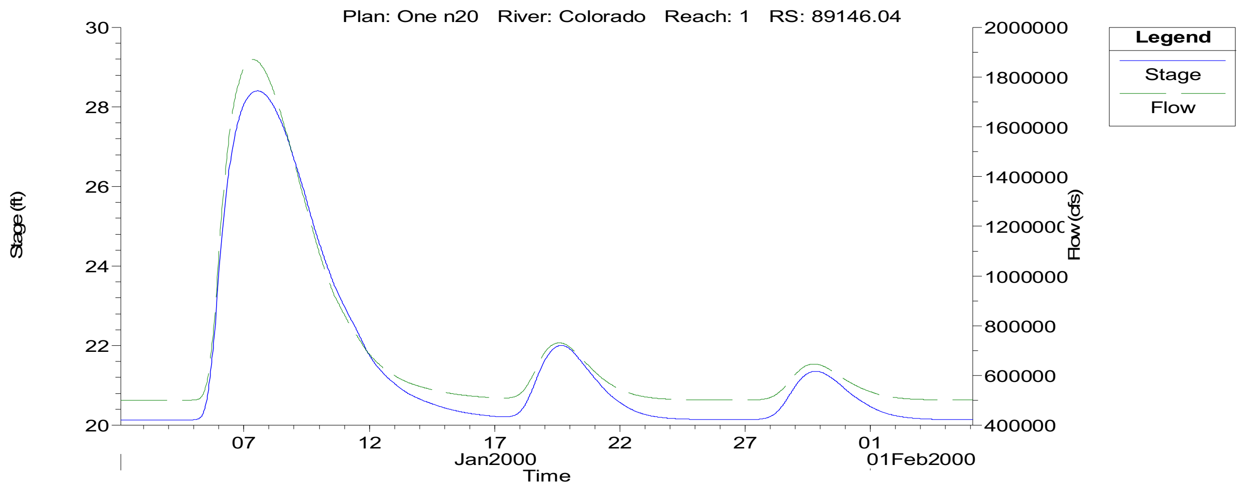


Figure 2.4S.4-9 Based Case Flood and Stage Hydrographs at the STP 3 & 4 Site

Note: Vertical Datum is NAVD 88; model start date was selected arbitrarily.

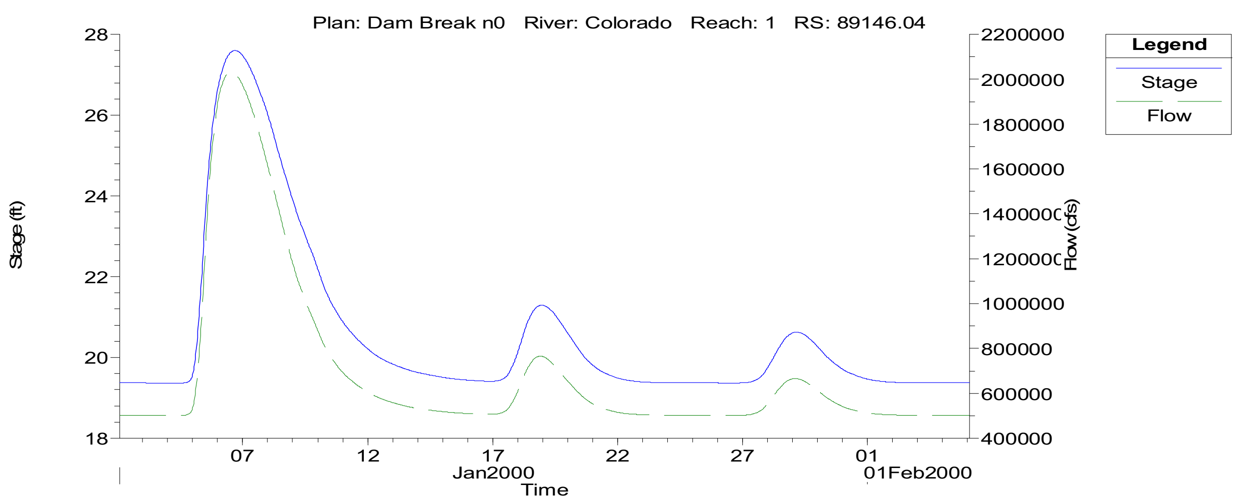


Figure 2.4S.4-10 Sensitivity Case Flood and Stage Hydrographs at the STP 3 & 4 Site

Note: Vertical Datum is NAVD 88; model start date was selected arbitrarily.

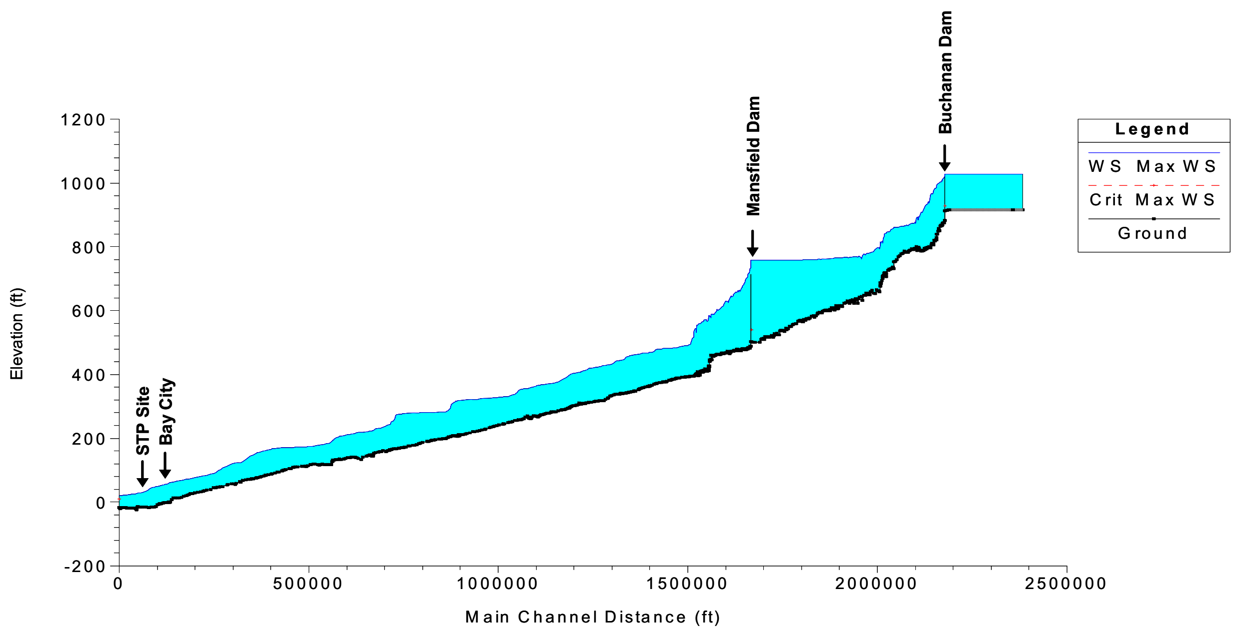


Figure 2.4S.4-11 Base Case Simulated Maximum Dam Break Surface Profiles from Buchanan Dam to 4,600 ft upstream of GIWW (Vertical Datum in NAVD 88)

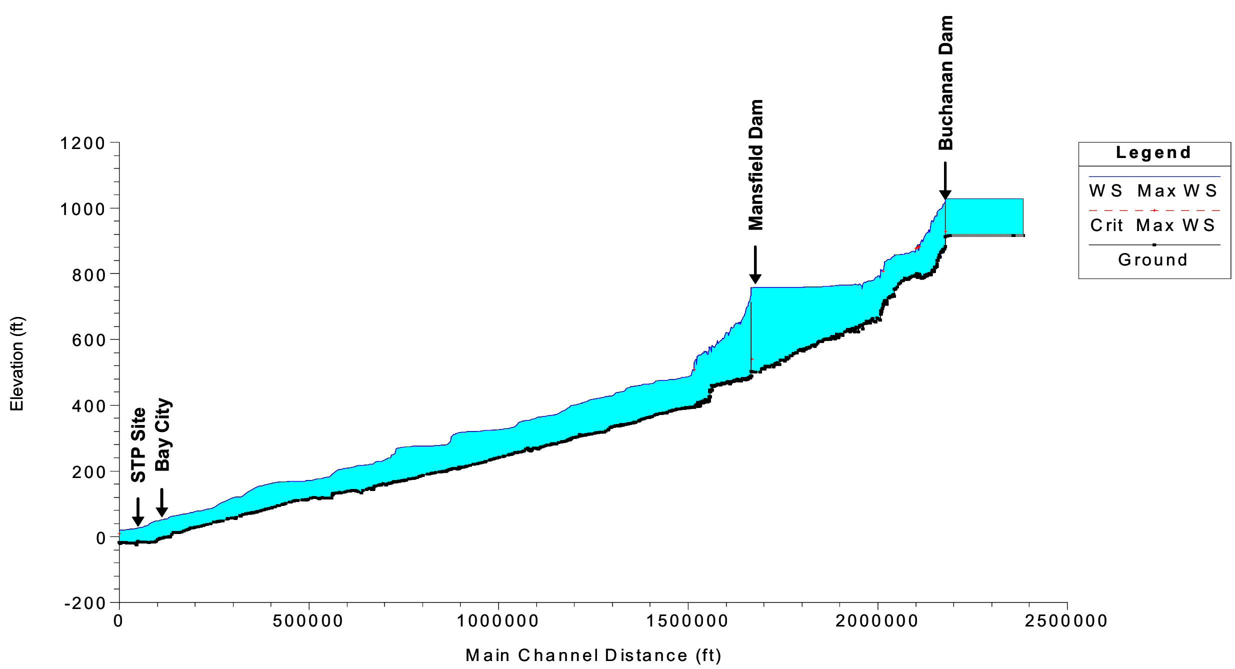


Figure 2.4S.4-12 Sensitivity Case Simulated Maximum Dam Break Surface Profiles from Buchanan Dam to 4600 ft Upstream of GIWW

Note: Vertical Datum in NAVD 88.

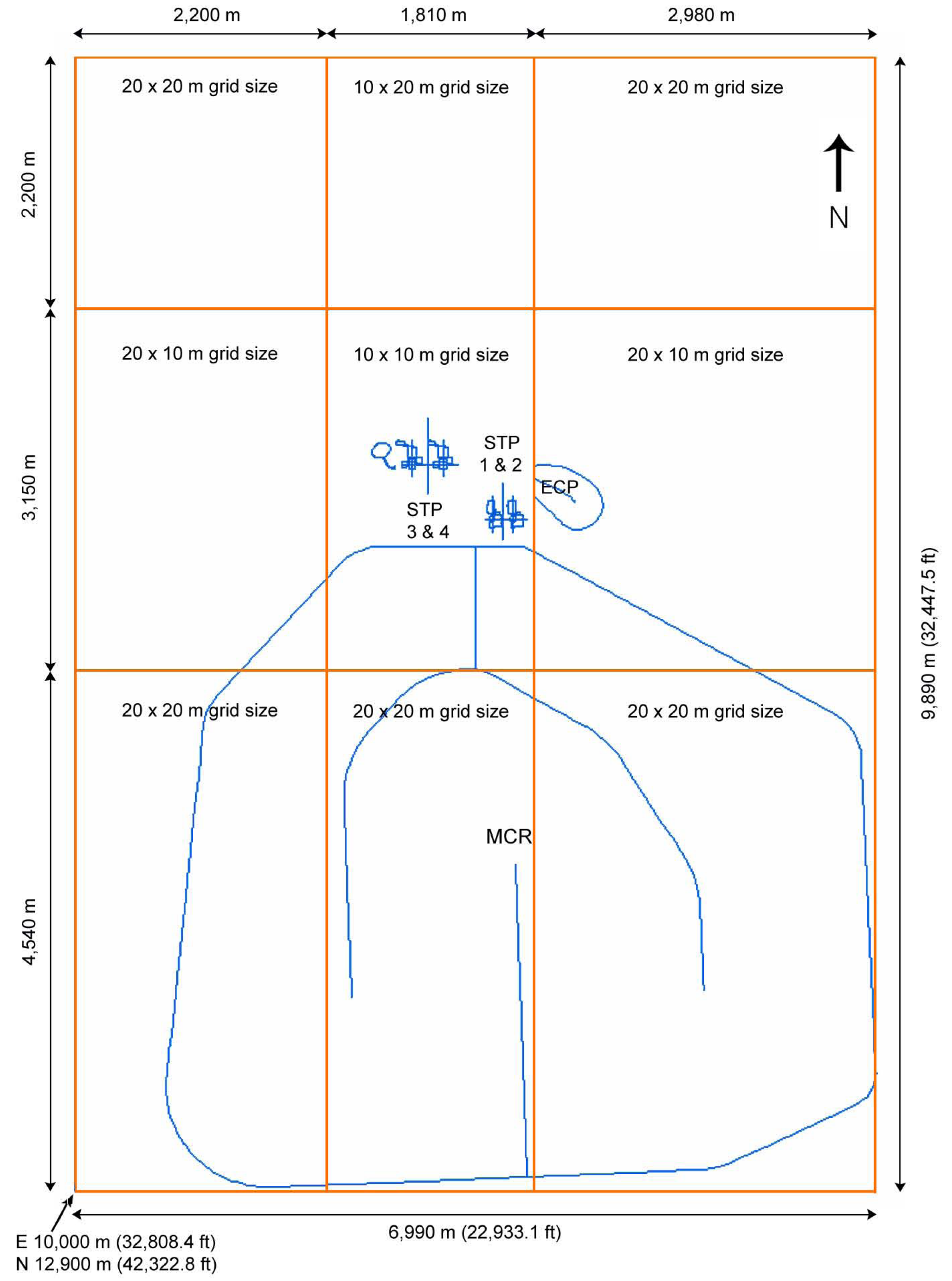


Figure 2.4S.4-13 Model Domain and Grid Sizes

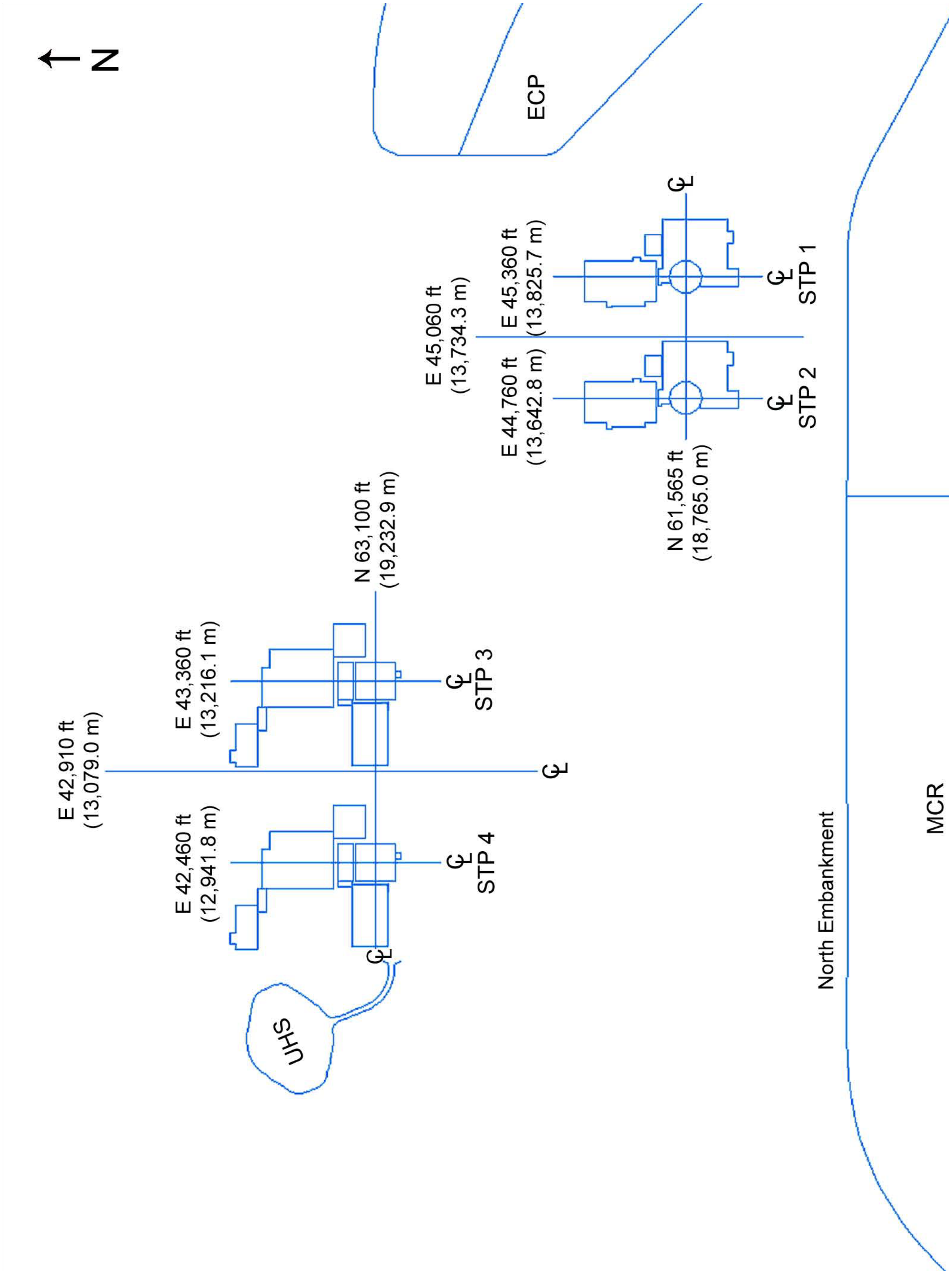


Figure 2.4S.4-14 Layout of the Safety Related Facilities in the Model

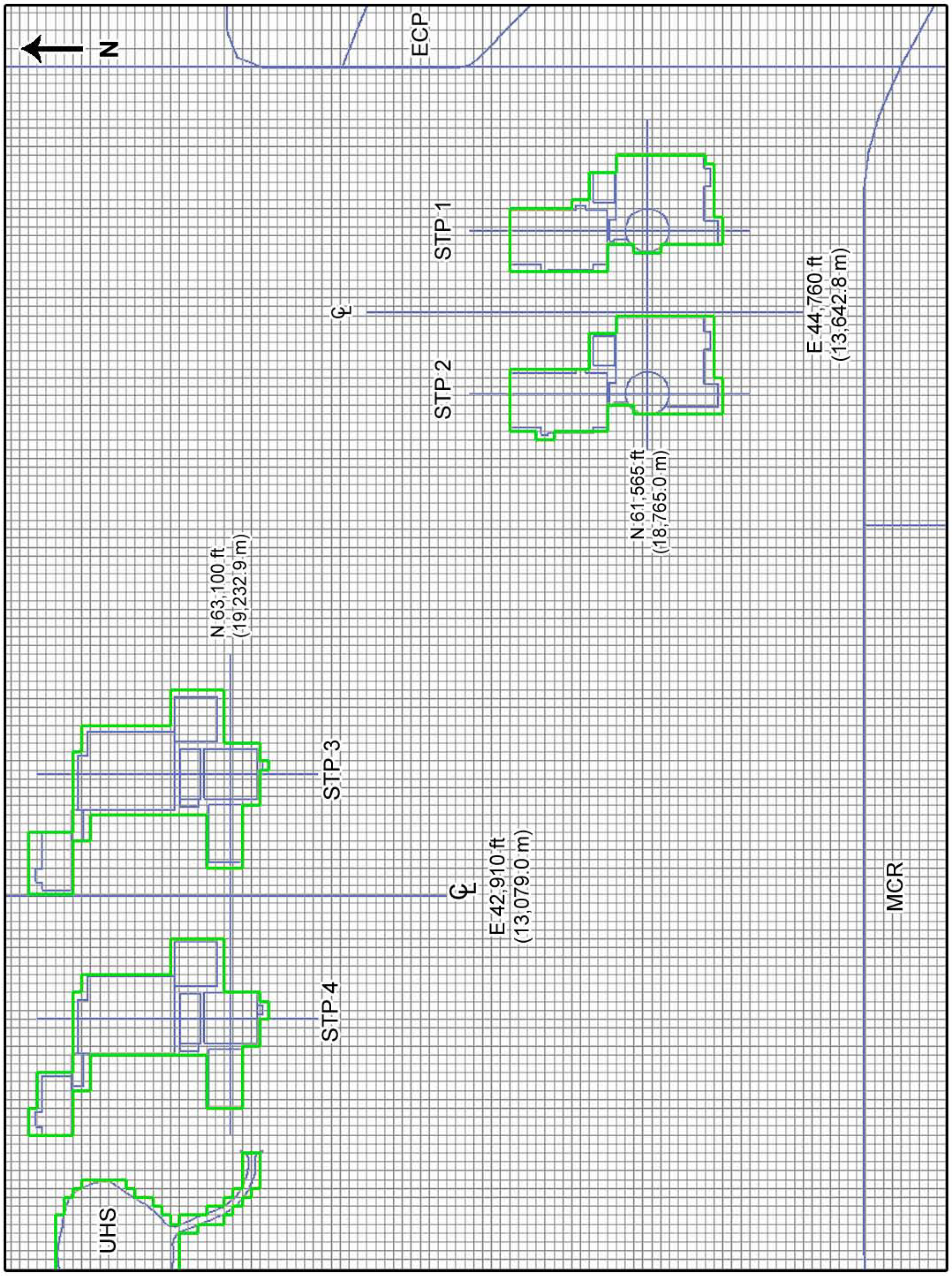


Figure 2.4S.4-15 Model Grid and Physical Building Outlines of STP 1 & 2 and STP 3 & 4

Note: Model outlines in green; physical building outlines in blue.

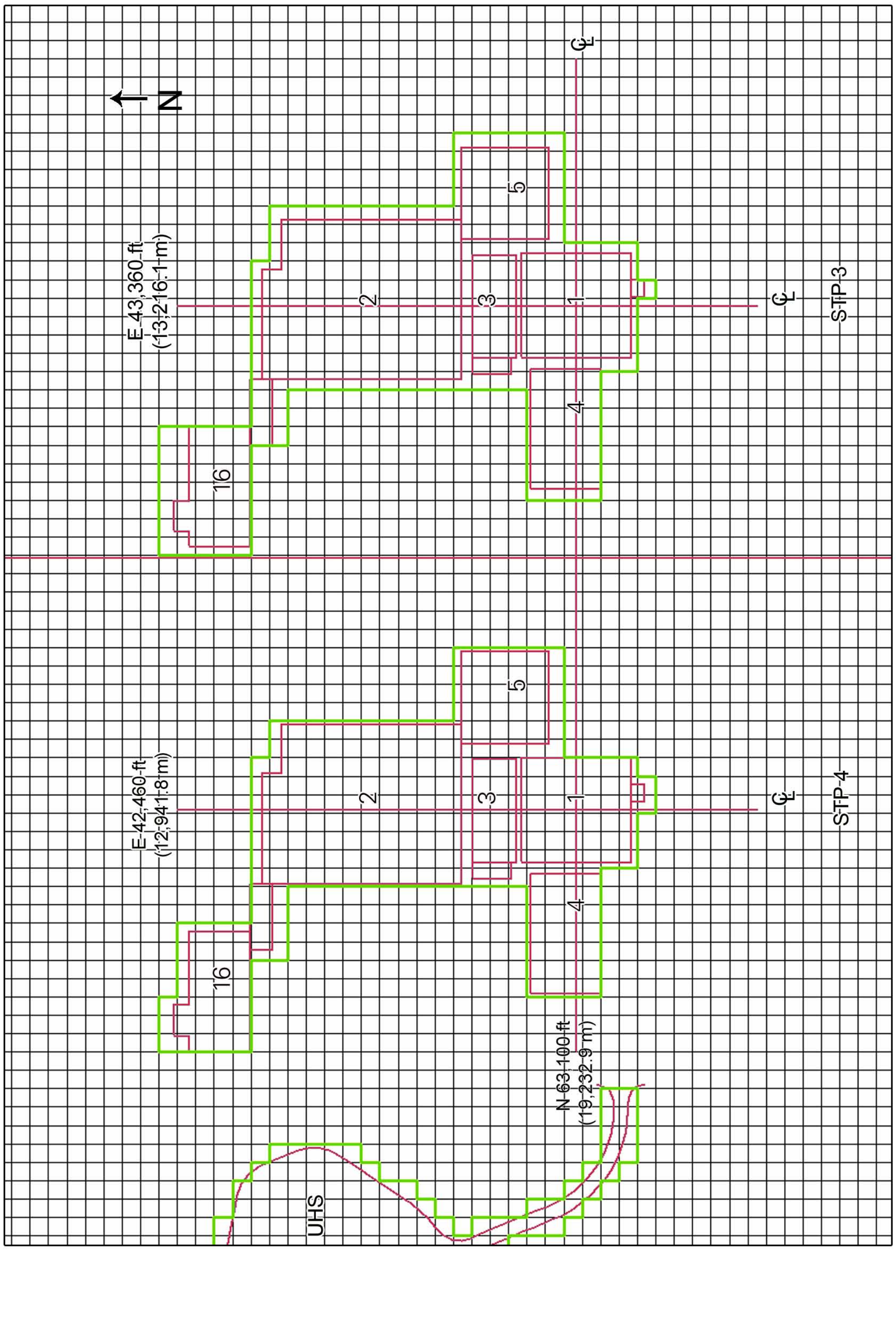
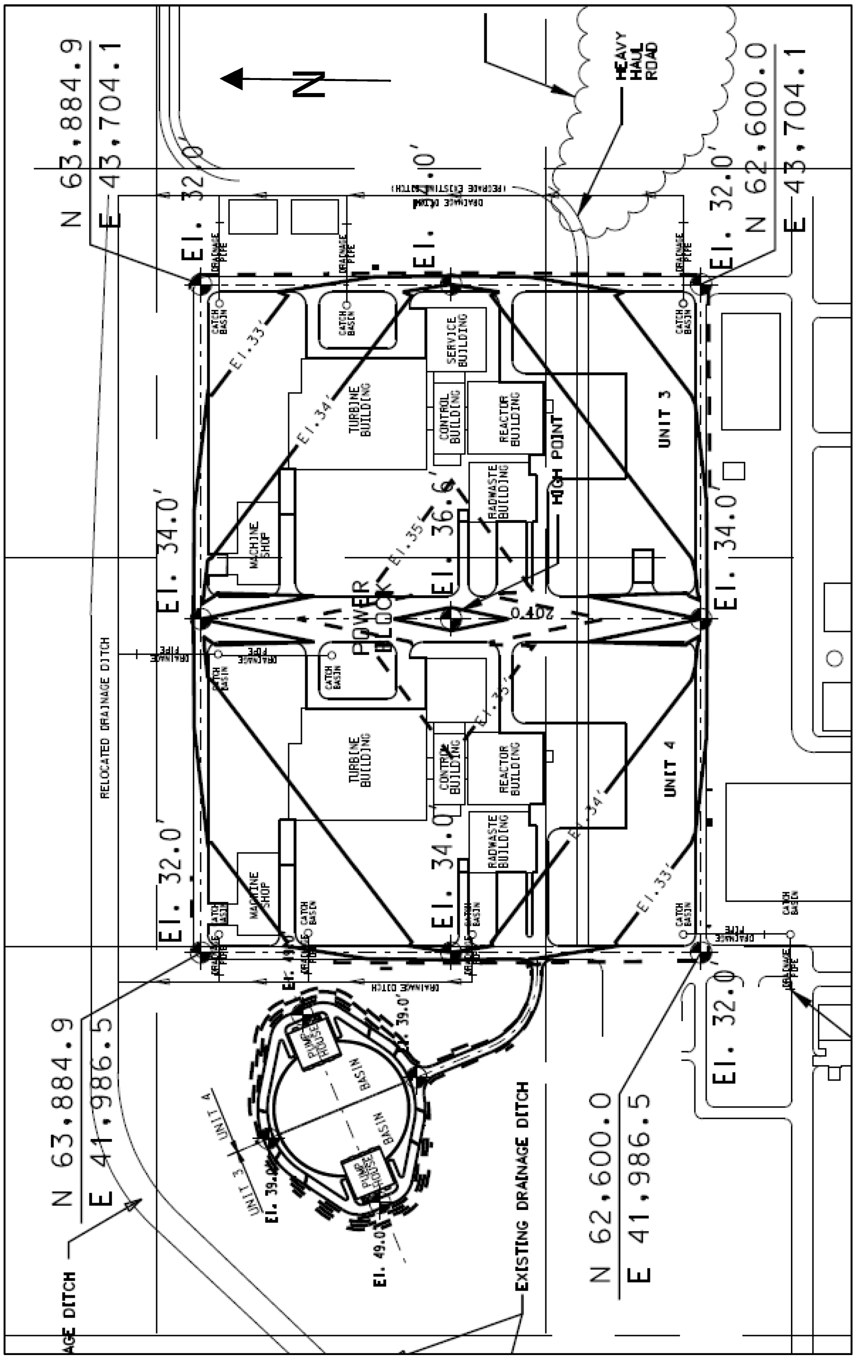


Figure 2.4S.4-16 Model Grid (10 m in Size) and Building Outlines of STP 3 & 4

Note: Model outlines in green; physical building outlines in blue.



Note: Eastings and Northings are in feet.

Figure 2.4S.4-17 Grading Plan of the Power Block of STP 3 & 4

Note: Eastings and Northings are in feet.

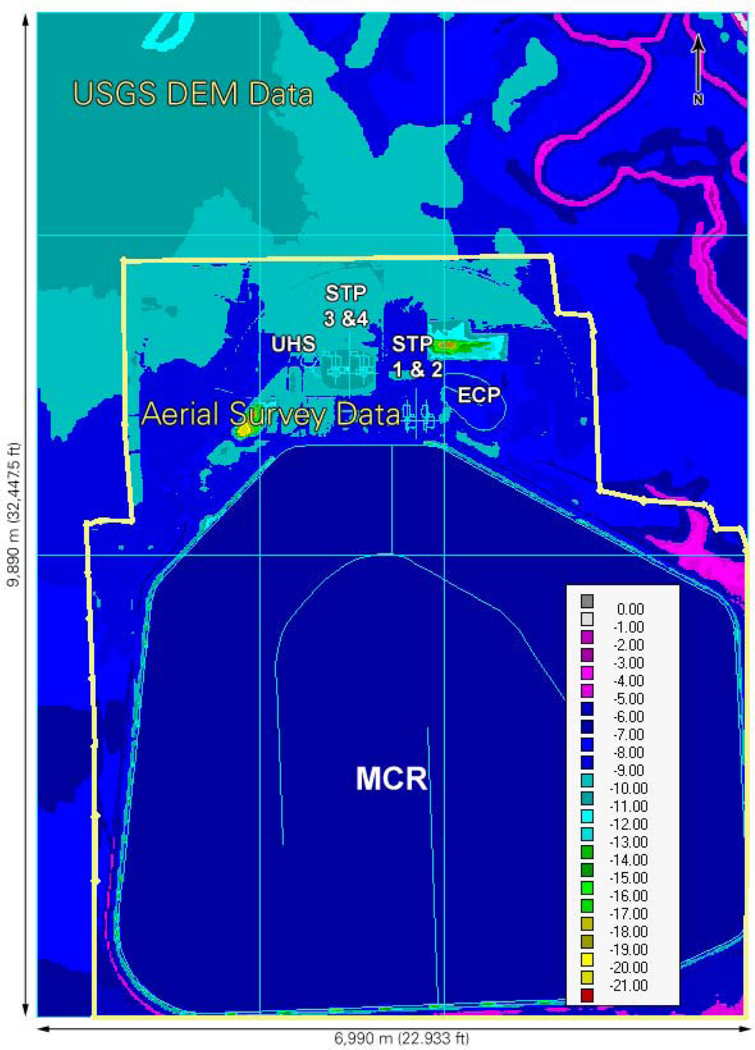


Figure 2.4S.4-18 Boundary Between the Aerial Survey Data and USGS DEM Data

Note: Bathymetry in Meters, referenced to MSL.

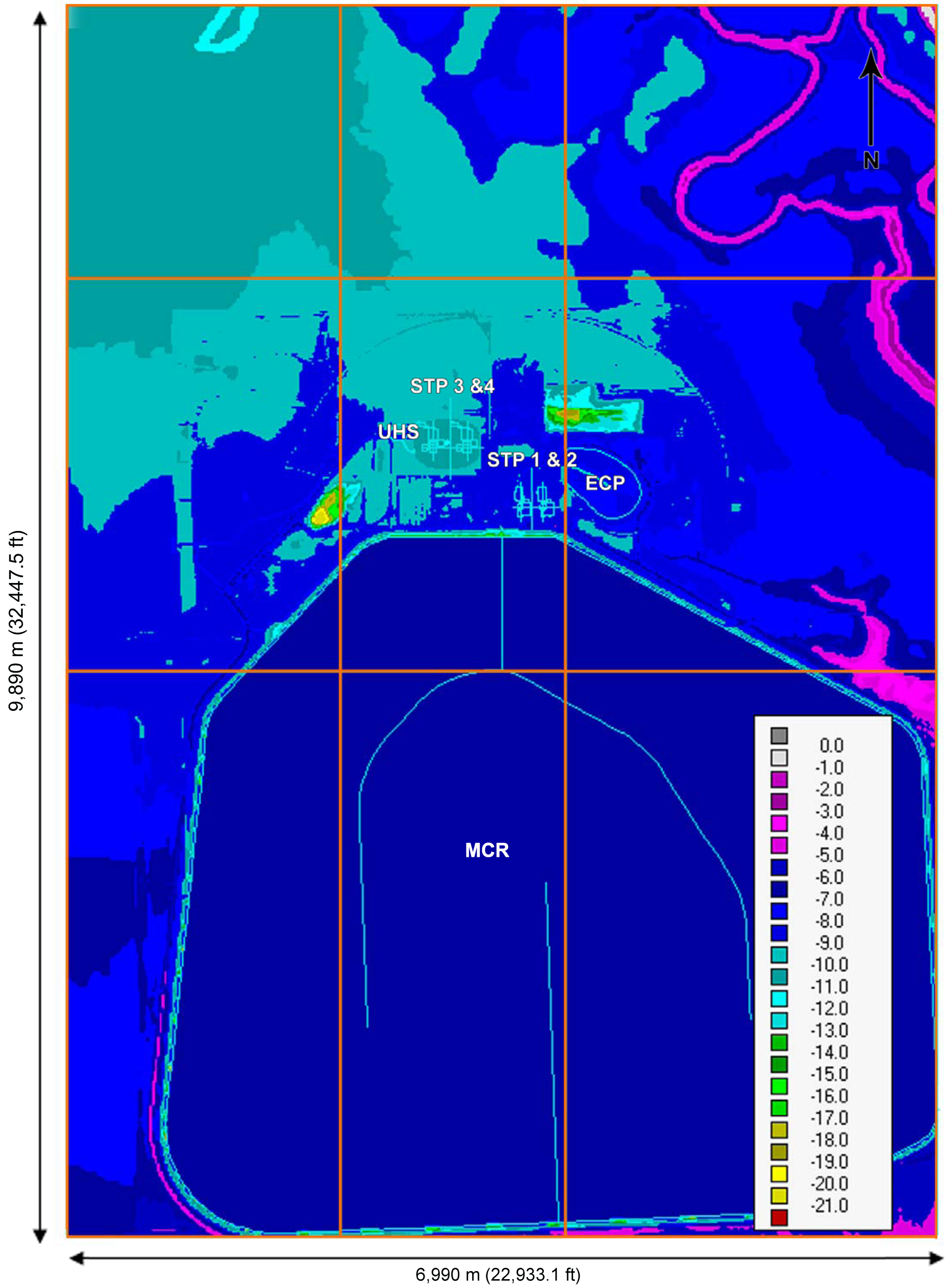


Figure 2.4S.4-19 Model Bathymetry

Note: In Meters, referenced to MSL.

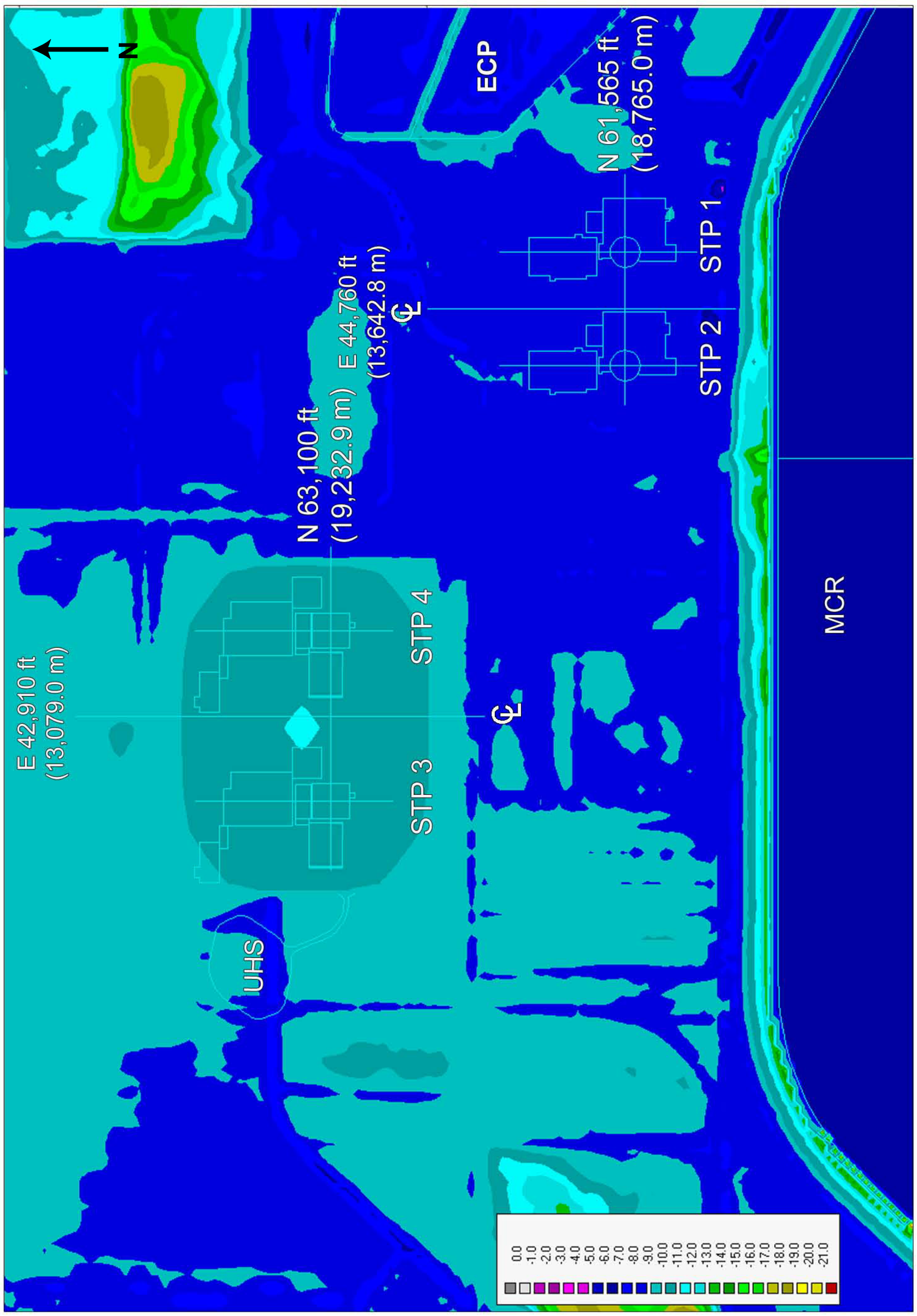


Figure 2.4S.4-20 Model Bathymetry Near the Power Blocks

Note: In Meters, referenced to MSL.

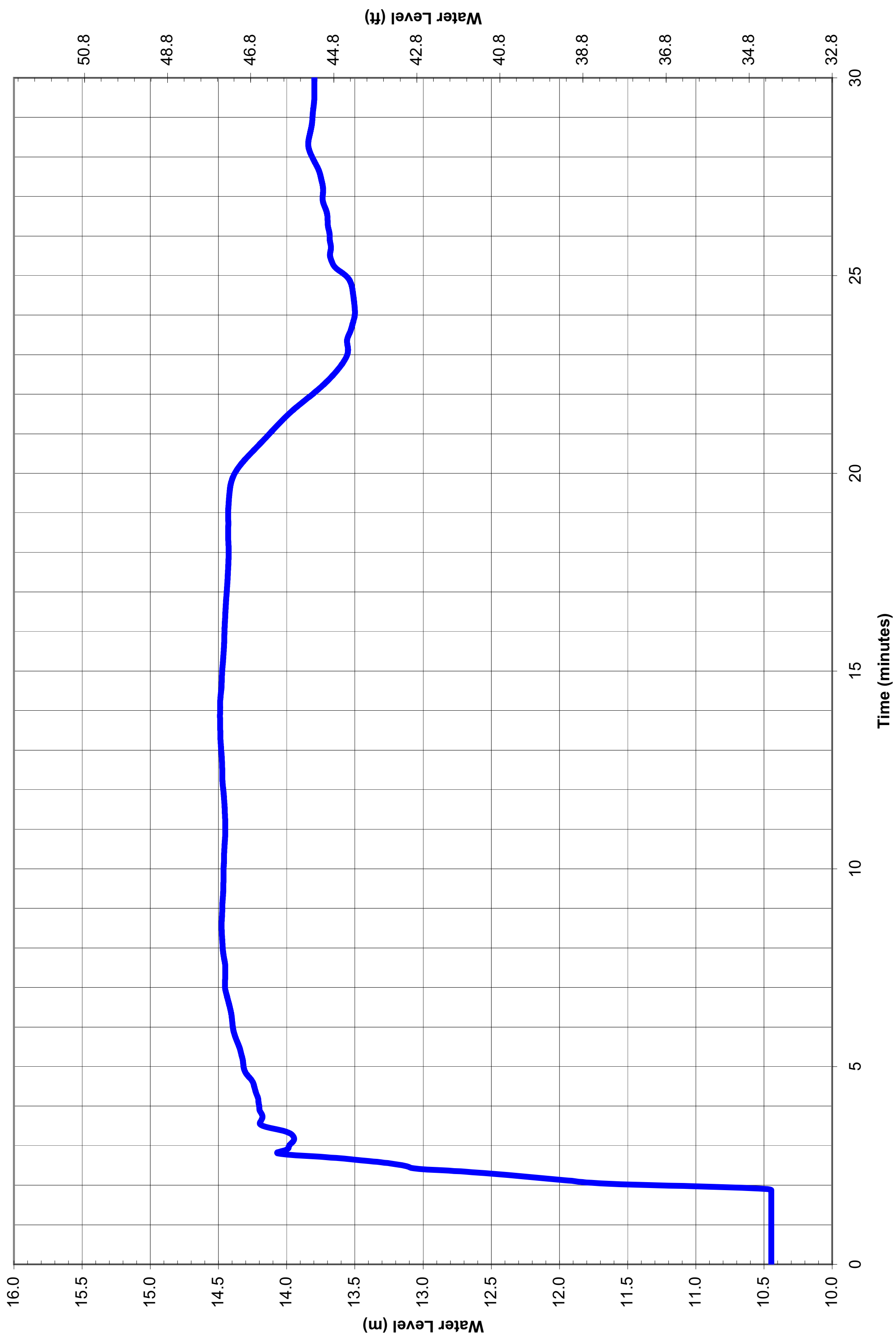


Figure 2.4S.4-21 Predicted Water Level Time History (MSL) at the Southern Face of STP 4 Reactor Building

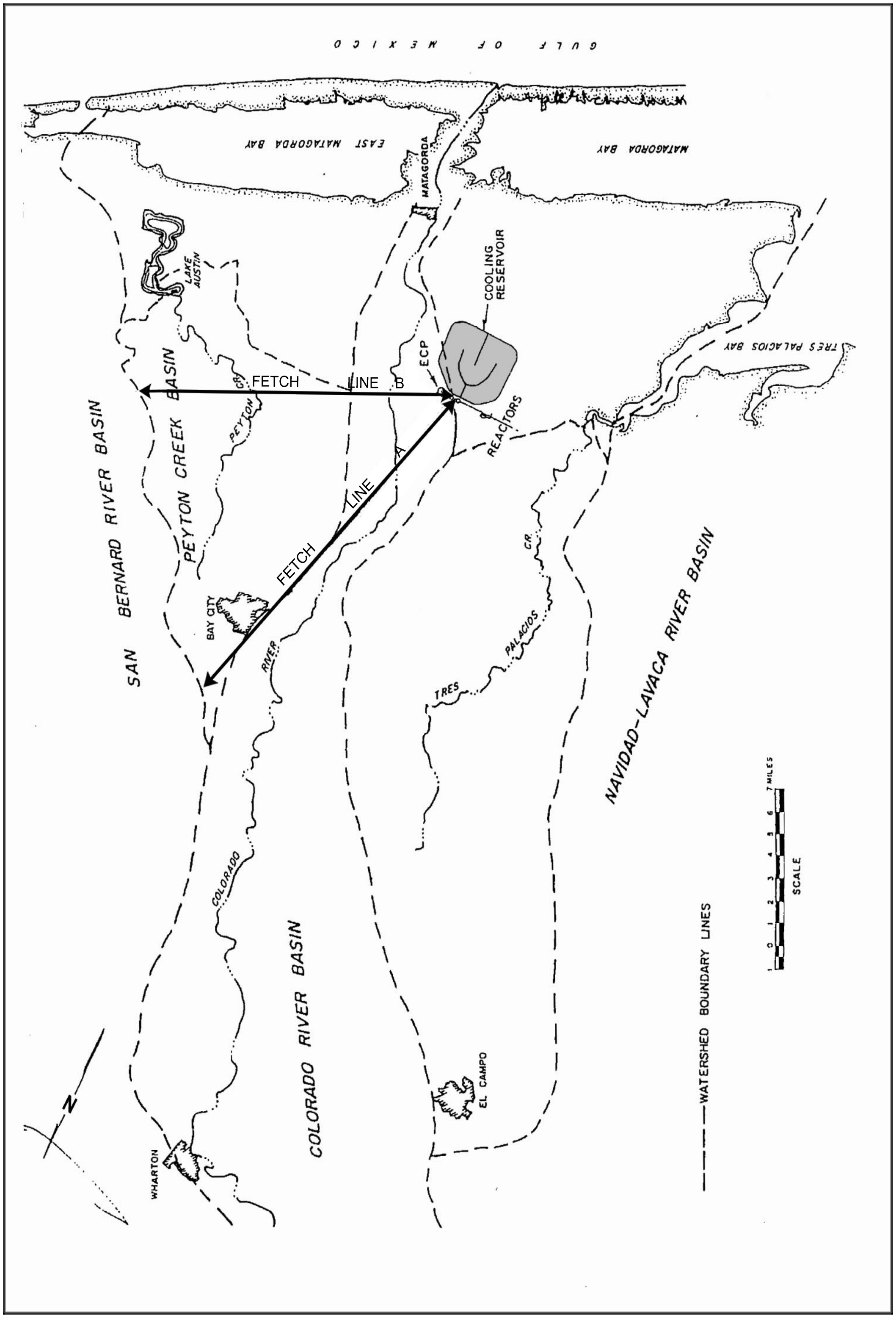


Figure 2.4S.4-22 Fetch Directions and Length