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 design basis flood level at STP 3 & 4 based on the worst case dam failure scenario, the postulated MCR embankment breach, was conservatively established as 40.0 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 interior reservoir side slopes of the MCR embankment are lined with 2 feet thick soil cement. 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 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 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, it is conceivable that a failure of the internal drainage system within the MCR embankment could saturate the embankment and allow seepage through it, which could then initiate a piping failure. Therefore, a piping failure of the MCR embankment was investigated and analyzed.

The northern MCR embankment, near the proposed circulating water intake and discharge pipeline, is the most critical location for piping failure because it is closest to, and inline with, Units 3 and 4. Two breach locations were considered for the analysis, one immediately east and one immediately west of the circulating water pipeline. Further discussion of breach parameter selection is presented in Subsection 2.4S.4.2.2.2.2.

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 embankment 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 embankment 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). Increasing the Manning's *n* values increases the simulated water levels because of increased roughness and therefore is a conservative approach in estimating the maximum flooding water levels at the plant site.

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 Embankment Breach Analysis

FLDWAV, a computer program developed by the National Weather Service (Reference 2.4S.4-12), was used to generate the outflow flood hydrograph from the MCR embankment breach, based on breach parameters discussed in Subsection 2.4S.4.2.2.2.2. This flood hydrograph was used as input to the two-dimensional flow model downstream of the breach.

RMA2 is a two-dimensional (2-D), depth-averaged finite-element hydrodynamic numerical model developed by the United States Army Corps of Engineers (USACE) (Reference 2.4S.4-12a). RMA2 was used to determine the flood elevations and velocities at the safety-related facilities of STP Units 3 and 4. The computer program can simulate dynamic water surface elevations and horizontal velocity components for subcritical, free-surface flow in a 2-dimensional flow field. The governing equations of RMA2 are the depth-integrated equations of fluid mass and momentum conservation in two horizontal directions. The governing equations are solved by finite-element method using the Galerkin Method of weighted residuals, and the integration in space is performed by Gaussian integration. Derivatives in time are replaced by a nonlinear finite difference approximation. The solution is fully implicit and the set of simultaneous equations is solved by the Newton-Raphson nonlinear iteration scheme. The computer code executes the solution by means of a front-type solver, which assembles a portion of the matrix and solves it before assembling the next portion of the matrix.

A 2-D model grid was developed based on topographic information and assigned parameters, such as Manning's roughness coefficient. Breach characteristics and a breach outflow hydrograph were incorporated into the 2-D grid, based on the breach analysis and FLDWAV results. A sensitivity analysis was conducted to evaluate the RMA2 results.

RMA2 does not have sediment transport modeling capability, and therefore, SED2D computer model (Reference 2.4S.4-12b) was used to conduct sediment transport simulation using RMA2 results as the driving hydrodynamics. The SED2D model, developed by the USACE, included a dynamic inflow load of sediments that was developed based on the breach erosion and sediment load analysis. The SED2D results were then evaluated for sediment concentrations and deposition depths at any given location. The Surface Water Modeling System (SMS) (Reference 2.4S.4-12c) was used as the pre- and post-processor for RMA2 and SED2D models.

2.4S.4.2.2.1 Assumptions in the MCR Embankment Breach Analysis

The following assumptions were used for the MCR embankment breach analysis:

(1) For modeling the flood elevation on the site, it was assumed that the large concrete structures such as STP Units 1 and 2 as well as Units 3 and 4, and several other tall and durable structures would remain in place during the flood. Other structures, such as metal skin buildings and warehouses, were assumed to be removed by the high velocity flood flow but have steel framing and associated remaining debris that would result in higher friction to flow. This higher friction to flow was incorporated by using a higher Manning's *n* for those elements.

(2) The bottom elevation of the MCR ranges approximately between elevations 16.0 ft and 28.0 ft. It was assumed that the average bottom elevation of the MCR is 20 ft (6.1 m), which is a representative low bed level in MCR.

(3) Breach side slopes were assumed to be 1 vertical to 1 horizontal for FLDWAV modeling.

(4) During the breach simulation it was assumed that there was no rainfall and therefore, there was no inflow to the MCR.

(5) It was assumed that the lateral expansion of the breach would occur symmetrically about its centerline.

2.4S.4.2.2.2 FLDWAV Flow Model Simulation

2.4S.4.2.2.2.1 Initial (Starting) Water Level in the MCR

The starting water level in the MCR considered for the breach analysis was 50.9 feet. This level corresponds to the response of the MCR to one-half PMP on the normal maximum operating level plus the effect of wind set-up produced by the 2-year wind speed (50 mph) from the south (Reference 2.4S.4-7).

2.4S.4.2.2.2.2 Selection of the MCR Embankment Breach Parameters

Reference 2.4S.4-12d by the Dam Safety Office of the U.S. Bureau of Reclamation describes several dam failure case studies that support empirical breach parameter relationships, and is considered the most complete and knowledgeable source for estimation of dam breach parameters. The breach parameters for the MCR embankment breach analysis were established based on discussions within this reference.

The portion of the northern embankment in line with and due south of Units 3 and 4 is the closest to the units, and therefore is considered the most critical location for a breach of the MCR embankment, with respect to flooding at STP 3 & 4. The top elevation of the embankment in this area is approximately El. 65.75 ft. A service road runs along the toe of the exterior slope of the MCR northern embankment. Due to an anticipated large scour hole that would occur at the breach location, it was assumed that the road would be eroded. The terrain immediately downstream of the road is considered to be the control for the breach bottom elevation. Therefore the breach bottom elevation was taken to be at El. 29 ft. Breach side slopes were taken to be 1 horizontal to 1 vertical, a ratio consistent with observations for earth-filled structures described in Reference 2.4S.4-12d.

Empirical relationships presented in Reference 2.4S.4-12d were used to determine the breach parameters consisting of (1) breach width; (2) time to failure; and (3) estimated peak flow from the breach, which was compared later to the peak flow from the breach resulting from FLDWAV modeling. Table 2.4S.4-5 presents empirical equations from Reference 2.4S.4-12d and the resulting breach parameters.

From Table 2.4S.4-5, it can be seen that Froehlich's equation yields the largest breach width estimate of all methods presented in Reference 2.4S.4-12d. Therefore, the Froehlich equation was used to estimate the breach width because it provides a conservative result in comparison with observed dam failures. Given the trapezoidal geometry of the breach, the average breach width of 417 feet yields a bottom breach width of 380 feet (417 - 2(65.75 -29)/2), which was used for FLDWAV embankment breach modeling.

The breach parameters estimated for the MCR embankment were also compared with the Teton Dam breach parameters, obtained from Reference 2.4S.4-12d. Teton Dam had more volume (310,000,000 m^3) and a greater breach height, h_b (86.9 m), which would allow significantly greater erosion to take place in creating the breach width. Froehlich's equation predicts an average breach width of 220 m (722 ft) for the Teton Dam. However, the actual average breach width of Teton Dam at failure was only 151 m (495 ft). Thus, Froehlich's equation over-predicts the breach width; therefore, the breach width determined for the MCR embankment using Froehlich's equation is considered conservative.

Time to failure, presented in Table 2.4S.4-5, was based on the equation given by MacDonald and Langridge-Monopolis (Reference 2.4S.4-12d). The breach width erosion rate (380/2 = 190 ft in 1.7 hours) is 112 feet per hour, assuming erosion opens equally to right and left of centerline. In comparison, the Teton Dam displayed a fairly rapid embankment erosion rate (496/2 = 248 ft in 1.25 hours) of about 200 feet per hour (Reference 2.4S.4-12d). This rapid rate was due to the higher hydraulic depth at the time of failure, which provides more energy to drive breach propagation. The water depth in Teton Dam was more than 200 ft, whereas the water depth in the MCR is less than 30 ft. Therefore, the estimated time to failure and breach width for the MCR embankment breach are considered reasonable.

Reference 2.4S.4-12d states that the Froehlich equation as shown in Table 2.4S.4-5 is one of the better available methods for prediction of peak breach discharge, because it correlates well with observed dam failure peak flow rates. The peak discharge estimated from that equation is 62,600 cfs.

2.4S.4.2.2.2.3 MCR Embankment Breach Outflow Hydrograph

The outflow hydrograph from the MCR embankment breach, generated by FLDWAV based on the aforementioned initial conditions and breach parameters is presented in Table 2.4S.4-6. The peak breach outflow predicted by FLDWAV is 130,000 cfs, whereas Froehlich's equation estimates 62,600 cfs. The relationship of estimated peak discharges associated with the respective hydraulic head at time of failure from Reference 2.4S.4-12e is given in Figure 2.4S.4-13. From this figure, the peak flow for the MCR embankment breach is only 20,000 cfs, compared to 130,000 cfs as determined by the FLDWAV program. Therefore, the outflow hydrograph with a peak outflow of 130,000 cfs used in the breach analysis is conservative.

2.4S.4.2.2.3 RMA2 Two-Dimensional Model Simulation

2.4S.4.2.2.3.1 Bathymetry Elevations and Two-Dimensional Grid Development

The topography of the STP site was used to determine model bathymetry for routing the flood flow resulting from the MCR embankment breach. The 2-D grid was developed using: (1) STP Site Topography; (2) STP Units 3 and 4 Site Grading Plan; and (3) STP Units 3 and 4 Plot Plan. The grading plan around Units 3 and 4 power block site is shown in Figure 2.4S.4-14. The grade elevation at the center of the power block is EL. 36.6 ft and slopes to El. 32 ft at the four corners. Facilities included in the model grid are the Reactor, Turbine, Control, Radwaste, Service and Hot Machine Shop buildings for Units 1 through 4. The Ultimate Heat Sinks for Units 3 and 4 and Essential Cooling Pond (ECP) for Units 1 and 2 were also included in the model grid.

The datums of the 2-D grid are in NAD 27 State Plane Texas South Central for the horizontal datum and NGVD 29 for the vertical datum. The northern embankment of the MCR was selected as the southern boundary of the 2-D grid, and road FM 521 was chosen as the northern boundary of the grid. The western and eastern boundaries of the grid were selected to be sufficiently far from Units 3 and 4 so the target area is not impacted by the model boundaries (Figure 2.4S.4-15). To assist the 2-D model stability and to further ensure that the target area is not impacted by model boundaries, a hypothetical sump was modeled along the east, north, and west boundaries of the developed 2-D grid. The use of the sump to help with model stability is a common practice in the 2-D modeling field, and the sensitivity analysis described below indicates that the hypothetical sump has no impact on model results in and around Units 3 and 4. As a result, the developed 2-D grid (excluding the artificial sump area) covers an area of 1,477 acres: 5,873 ft in the north-south direction, and 12,455 ft in the east-west direction. Figures 2.4S.4-16 and 2.4S.4-17 show the 2-D grid with elevations for the east breach and west breach, respectively. The 2-D grid includes 2,348 nodes and 1,088 elements. The size and location of these elements were selected to best represent physical features, particularly around Units 3 and 4. The areas of the 2-D elements range from about 2,500 square feet near the reactor buildings to about 144,000 square feet away from the units.

2.4S.4.2.2.3.2 Manning's Roughness Coefficients

The Manning's roughness coefficient (*n* value) for each model element was assigned based on typical values published by the United States Geological Survey (USGS) (References 2.4S.4-12f and 2.4S.4-12g) and the HEC-RAS manual (Reference 2.4S.4-12h). Each major building was evaluated on whether it would remain in place following the flood caused by a MCR embankment breach. Those buildings that were assumed to remain in place were considered "hard buildings." Any hard buildings higher than elevation 62 feet were considered to be a total blockage to the flow, and therefore were shown as blank areas in the 2-D grid. Those buildings assumed to fail were considered "soft buildings." Soft buildings were assumed to be destroyed with foundation slab remaining in the grid. These buildings were considered "high drag" areas with a higher roughness value to represent the effects of remaining frame and debris. Any buildings not included in the 2-D grid were represented by a higher Manning's n value. Due to the resolution of the grid, the Vehicle Barrier System around the power blocks was not built into the grid, but instead was represented by higher Manning's *n* value. Manning's n values assigned to each material type are listed in Table 2.4S.4-7. Figure 2.4S.4-18 shows the material types assigned to various elements in the 2-D grid. These Manning's *n* values were conservatively determined for each type of surface.

2.4S.4.2.2.3.3 Boundary Conditions

The downstream boundaries of the model were positioned far enough downstream so that the maximum flood level at the STP Units 3 and 4 safety-related buildings due to a MCR embankment breach would occur before the flood front reaches the two boundaries. A constant water surface elevation was defined for the downstream boundary condition. A sensitivity analysis was performed on the downstream boundary condition, as discussed in Subsection 2.4S.4.2.2.4.1.

2.4S.4.2.2.4 Results of MCR Embankment Breach Analysis

2.4S.4.2.2.4.1 Water Levels and Velocities

Critical STP 3 and 4 site locations for RMA2 model results are shown on Figure 2.4S.4-19. The variation in water surface elevation at these locations from 1.2 hours to 2.5 hours of the model simulation are presented in Figures 2.4S.4-20 and 2.4S.4-21 for the east breach and west breach, respectively. This selected period includes the peak water level and peak velocity near the plant buildings. The peak water level of 38.8 feet occurred at the Unit 4 Ultimate Heat Sink structure for the west breach scenario. Peak water surface elevations for the east breach and west breach are shown on the plan grid in Figures 2.4S.4-21(a) and 2.4S.4-21(b), respectively. Peak velocities associated with the east breach and west breach are shown in Figures 2.4S.4-21(c) and 2.4S.4-21(d), respectively. The maximum velocity of the flood flow was found to be 4.72 feet per second and occurred between Units 3 and 4 (point 8 on Figure 2.4S.4-19). The variation in velocity at locations 1 through 8 for the period containing peak velocities for the east and west breach scenarios is shown in Figures 2.4S.4-21(e) and 2.4S.4-21(f), respectively.

A sensitivity analysis was conducted to determine the effect of boundary condition on the resulting water levels. The analysis indicated that changing the water surface elevation at the downstream boundary from 32.5 feet to 34 feet does not affect the peak flood levels for the site.

2.4S.4.2.2.4.2 Effects of Sedimentation and Erosion

The MCR embankment breach analysis also considered the material eroded during the breach. The embankment material eroded is comprised mostly of clay, with a small percentage of sand from the internal drainage system and soil cement from the interior embankment slope lining. The erosion process will also produce a scour hole downstream of the breach that extends below the breach bottom elevation. The dimensions of this scour hole, based on lab results from Reference 2.4S.4-12i, are estimated to be 20 feet deep, 203 feet long and 380 feet wide. The scour hole contributes 1,543,000 cubic feet of clay to the flood flow. The material eroded from the MCR embankment contributes an additional 1,697,314 cubic feet of clay; 75,644 cubic feet of sand; and 117,562 cubic feet of soil cement. The flood flow from the MCR embankment breach would not erode the STP 3 and 4 plant site area because surfacing in this area is mostly concrete or asphalt pavement or compacted gravel and grass. The maximum velocity of 4.72 ft/s would not cause severe erosion of these surfaces, and any minor erosion around corners of the buildings would not impact the safety-related facilities of Units 3 and 4.

SED2D sediment modeling indicated some deposition on the outer edge of the model domain and little or no deposition within the STP site. These results are consistent with the sediment concentration results in that the majority of the clay and sand loads would be suspended in the flood flow and washed downstream, beyond the STP site. The soil cement lining on the interior wall of the embankment was not simulated. This material would likely enter the water as chunks or blocks as the embankment collapses, and these large concrete blocks would be carried only a short distance from the breach before settling to the bottom. The sediment loading would cease when the breach opening expansion ends; however, sediment-free high flow would continue for a long period afterwards until the water in MCR is totally emptied. This high flow period would prevent any remaining clay or sand particles from settling and would wash away any small depositions in the study area.

2.4S.4.2.2.4.3 Hydrodynamic Forces

The maximum water levels and velocities obtained near Units 3 and 4 were used to assess the hydrodynamic loadings on the plant buildings. Figures 2.4S.4-21(g) and 2.4S.4-21(h) show the time-dependent plots of the velocities at this location during the east and west breach scenarios, respectively. The peak velocities observed were 4.72 and 4.68 feet per second for the east and west breach scenarios, respectively. Figures 2.4S.4-21(g) and 2.4S.4-21(h) also show the sediment concentrations predicted by the SED2D model. The sediment-laden water density was used for hydrodynamic load calculations. The figures show that the sediment concentrations at the time and location of peak velocities would be 16.5 kg/m^3 and 15 kg/m^3 for the east and west breach scenarios, respectively. However, Figure 2.4S.4-21(g) shows a maximum concentration of 23 kg/m^3 occurring at approximately T = 1.3 hours. Conservatively, the maximum sediment concentration was used in conjunction with the maximum velocity to determine the hydrodynamic loads on the STP 3 and 4 plant facilities. Selecting a 23 kg/m^3 sediment concentration, a water density of 1023 kg/m^3 or 63.85 lb/ft^3 was used for load calculations. The maximum hydrostatic force on any plant building would be due to the depth of floodwater at the maximum water level. Hydrodynamic loads were calculated using the drag force formula with a drag coefficient conservatively set to 2.0, as presented below:

$$\text{Force (lb/ft}^2\text{)} = 2.0 \times \text{Density (lb/ft}^3\text{)} \times \text{Velocity}^2 \text{ (ft}^2\text{/sec}^2\text{)} / 2g$$

The maximum drag force due to the maximum velocity of flow near the plant buildings is estimated as 44 pounds per square foot of the projected submerged area of the buildings.

2.4S.4.2.2.4.4 Spatial Extent of Flooding Due To MCR Embankment Breach

For both the east and west MCR embankment breach scenarios flood water from the breach opening will flow through the area encompassing Units 1 and 2 and Units 3 and 4, and will spread into the area bounded by FM 521. This road has a top of road elevation of approximately 28 feet to 30 feet, as seen from the USGS topographic map of the area (Figure 2.4S.4-21(i)). North of FM 521 and west of the west MCR embankment there are levees with approximate top elevations of 29 feet to 30 feet. South of the MCR along its south embankment is an east - west canal with levees on both sides. The area around the STP plant has an approximate grade elevation varying from 25 feet to 30 feet.

The area around the STP plant slopes east towards the Colorado River. Therefore, most of the flood water from the breach would flow to the Colorado River. A portion of the breach flow will also reach the Little Robins Slough to the west, which flows south along the west MCR embankment. From there, the water will either flow east to the Colorado River or will flow under the east-west canal through existing siphons and may flow through several swampy areas to the intracoastal waterway.

It is unlikely that the breach flood water will overflow over FM 521 and west levees. If this happens, a small portion of the breach flood flow may reach the Tres Palacios River to the west of the STP site.

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 conservatively established as 40.0 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.

All the safety-related facilities including the UHS are located in the power block island. The power block island will have a grade elevation of approximately 34.0 ft near the plant buildings and will slope towards the periphery to an elevation of 32.0 ft at the edges. The outward slope of the island will be at 10H:1V from elevation 32.0 ft to an existing grade elevation of 28.0 ft.

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 using a slope of 10H:1V for the power block island, the wave run-up was estimated to be 1.9 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. 34.4 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 critical fetch.

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 design basis flood level at the power block and UHS of STP 3 & 4 based on the breaching of the MCR's northern embankment is at El. 40.0 ft MSL. Because the design basis flood level is higher than both the nominal plant grade of 34.0 ft MSL and 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 40.0 ft MSL. All ventilation openings of safety buildings are located at 40.0 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 and sedimentation during a MCR embankment breach event is discussed in Subsections 2.4S.4.2.2.4.2 and 2.4S.10.

2.4S.4.4 References

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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 MCR Embankment Breach Parameters and Peak Discharge Based on Empirical Equations from Reference 2.4S.4-12d

Parameter	Equation	Results
Time to Failure (hrs)	$t_f = 0.0179(0.0261(V{*}h_w)^{0.769})^{0.364}$	1.7 hours
Average Breach Width (m)	$B_{ave} = 0.1803\ V^{0.32}\ h_b^{0.19}$	127 m (417 ft)
Peak Q (m^3/s)	$Q_p = 0.607\ V^{0.295}\ h_w^{1.24}$	1172.8 m^3/s (62,600 cfs)

B_{ave} = average breach width
h_w = depth of water above breach in m = 50.9' – 29' = 21.9' = 6.7 m
h_b = the height of breach from the top of embankment in m = 66' – 29' = 37' = 11.3 m
V = volume of water in the MCR between El. 29' and El. 50.9' in m^3 = 188,400,000 m^3 (152,700 ac-ft)

Table 2.4S.4-6 MCR Embankment Breach Outflow Hydrograph

Time (hours)	Flow (cfs)	MCR Water Surface Elevation (ft)
0	0	50.90
0.1	1,100	50.90
0.2	3,970	50.89
0.3	8,570	50.88
0.4	15,500	50.87
0.5	24,700	50.85
0.6	30,600	50.82
0.7	37,200	50.78
0.8	47,300	50.73
0.9	54,700	50.68
1.0	79,600	50.59
1.1	85,900	50.49
1.2	92,300	50.40
1.3	100,700	50.28
1.4	108,500	50.15
1.5	116,100	50.03
1.6	123,500	49.88
1.7	130,000	49.74
1.8	126,700	49.58
1.9	124,500	49.46
2.0	122,600	49.29
2.1	120,800	49.13
2.2	119,000	49.00
2.3	117,400	48.86
2.4	115,600	48.70
2.5	113,900	48.56
3	112,800	47.88
6	83,150	44.44
9	63,030	41.86
12	48,890	39.88
15	38,680	38.32
18	31,110	37.08
21	25,390	36.07
24	21,000	35.24
27	17,560	34.56
30	14,840	33.98

Table 2.4S.4-7 Material Types and Associated Manning's *n*

Material Type	Manning's *n*
Water	0.030
Short Hard Building	0.100
Soft Building / High Drag	0.085
Vehicle Barrier Walls (VBW)	0.085
Gravel	0.035
Open Space	0.040
Concrete Slab	0.012
Road (Concrete)	0.013
Channel	0.040
Pipeline	0.100
Artificial Sump	0.100

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	28.6	3.9	1.9	34.4

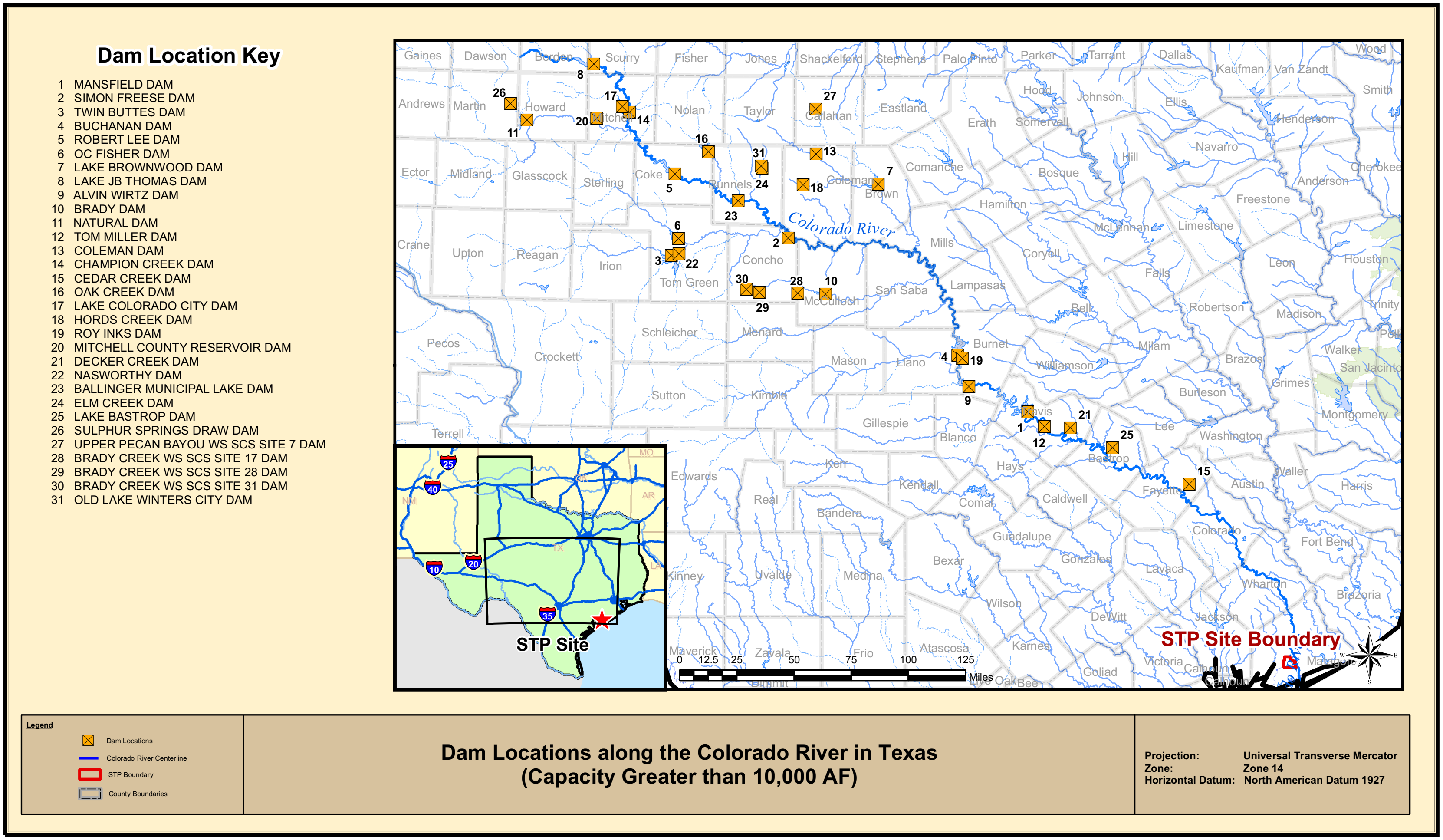
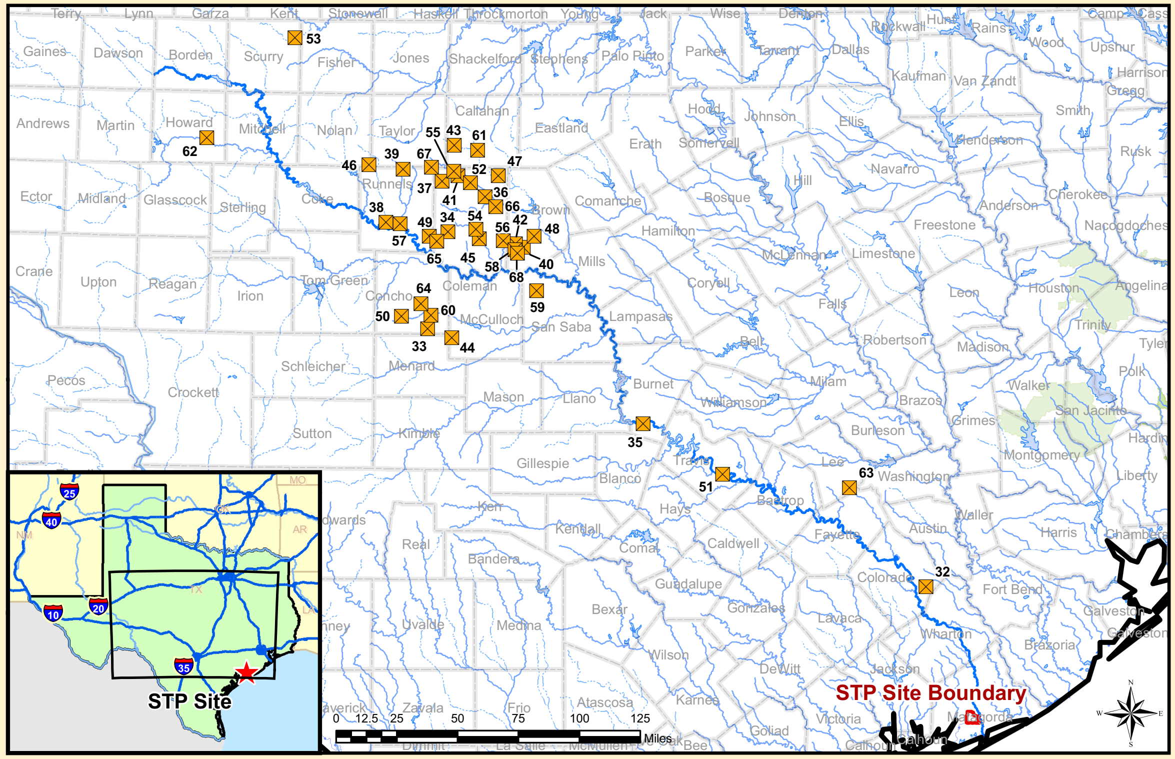


Figure 2.4S.4-1a Locations of Dams with Storage Capacity Over 10,000 AF in the Colorado River Basin Upstream of the STP 3 & 4 Site

Dam Location Key

32 EAGLE LAKE DAM
33 BRADY CREEK WS SCS SITE 20 DAM
34 NORTHWEST LATERALS WS SCS SITE 5A DAM
35 MAX STARCKE DAM
36 JIM NED CREEK WS SCS SITE 25 DAM
37 JIM NED CREEK WS SCS SITE 12E1 DAM
38 BALLINGER CITY LAKE DAM
39 ELM CREEK WS NRCS SITE 3 REV
40 CLEAR CREEK WS SCS SITE 6 DAM
41 JIM NED CREEK WS SCS SITE 21 DAM
42 CLEAR CREEK WS SCS SITE 4 DAM
43 UPPER PECAN BAYOU WS SCS SITE 2 DAM
44 BRADY CREEK WS SCS SITE 14 DAM
45 HOME CREEK WS SCS SITE 13 DAM
46 VALLEY CREEK WS SCS SITE 1 DAM
47 UPPER PECAN BAYOU WS SCS SITE 24 DAM
48 BROWNWOOD LATERALS WS SCS SITE 3 DAM
49 NORTHWEST LATERALS WS SCS SITE 1 DAM
50 BRADY CREEK WS SCS SITE 32 DAM
51 LONGHORN DAM
52 JIM NED CREEK WS SCS SITE 23 DAM
53 ELM CREEK WS NRCS SITE 7 REV
54 HOME CREEK WS SCS SITE 7A DAM
55 JIM NED CREEK WS SCS SITE 12 DAM
56 MUKEWATER CREEK WS SCS SITE 10A DAM
57 ELM CREEK LAKE DAM
58 CLEAR CREEK WS SCS SITE 3 DAM
59 SE LATERALS WS SCS SITE 7 DAM
60 BRADY CREEK WS SCS SITE 21 DAM
61 UPPER PECAN BAYOU WS SCS SITE 12 DAM
62 MOSS CREEK LAKE DAM
63 CUMMINS CREEK WS SCS SITE 1 DAM
64 BRADY CREEK WS SCS SITE 36 DAM
65 NORTHWEST LATERALS WS SCS SITE 2 DAM
66 JIM NED CREEK WS SCS SITE 26A DAM
67 JIM NED CREEK WS SCS SITE 19 DAM
68 CLEAR CREEK WS SCS SITE 1 DAM



Legend
Dam Locations
Colorado River Centerline
STP Boundary
County Boundaries

Dam Locations along the Colorado River in Texas
(Capacity Between 5,000 and 10,000 AF)

Projection: Universal Transverse Mercator
Zone: Zone 14
Horizontal Datum: North American Datum 1927

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

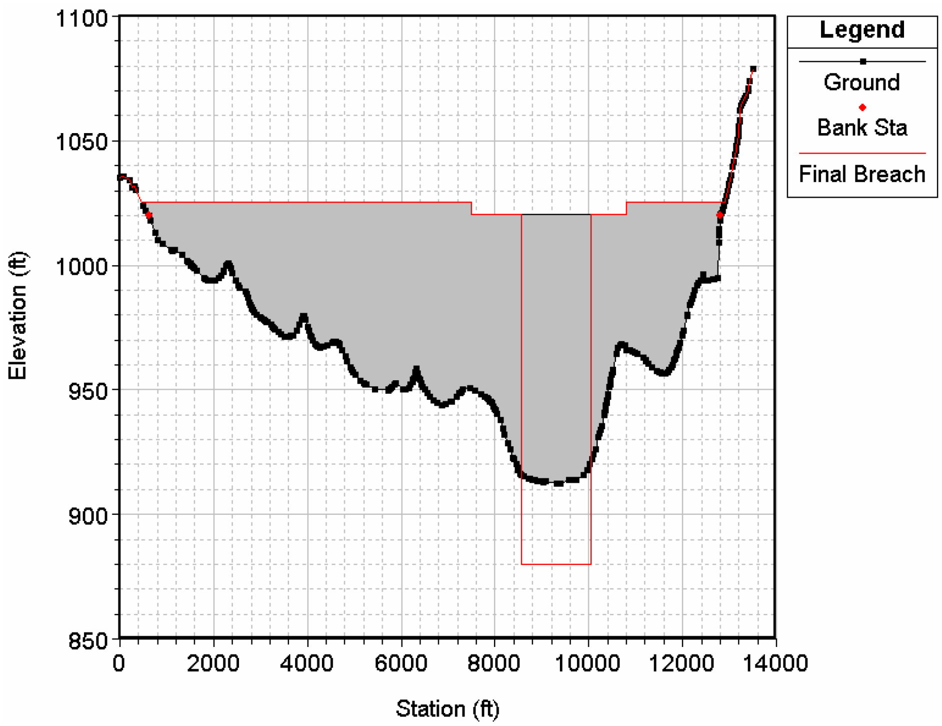


Figure 2.4S.4-2 Model Cross Section at Buchanan Dam

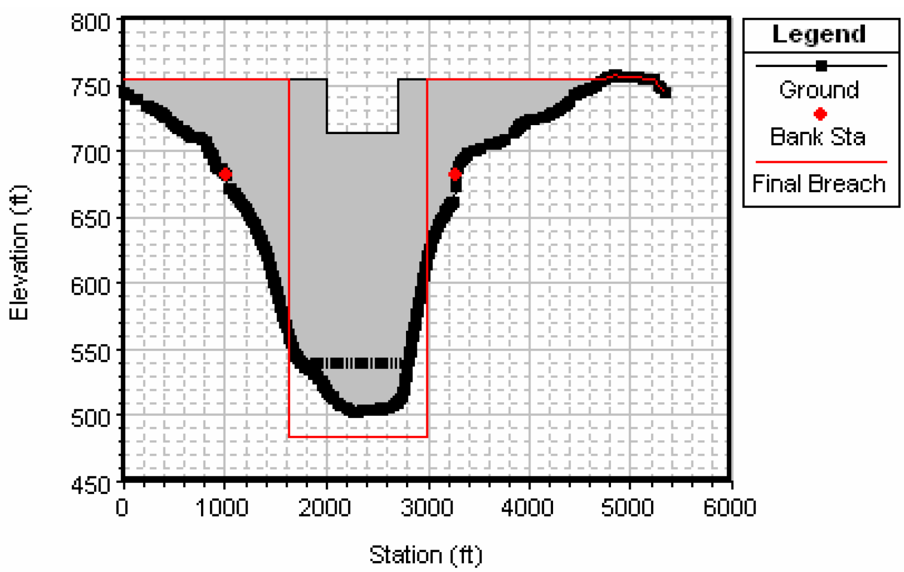


Figure 2.4S.4-3 Model Cross Section at Mansfield Dam

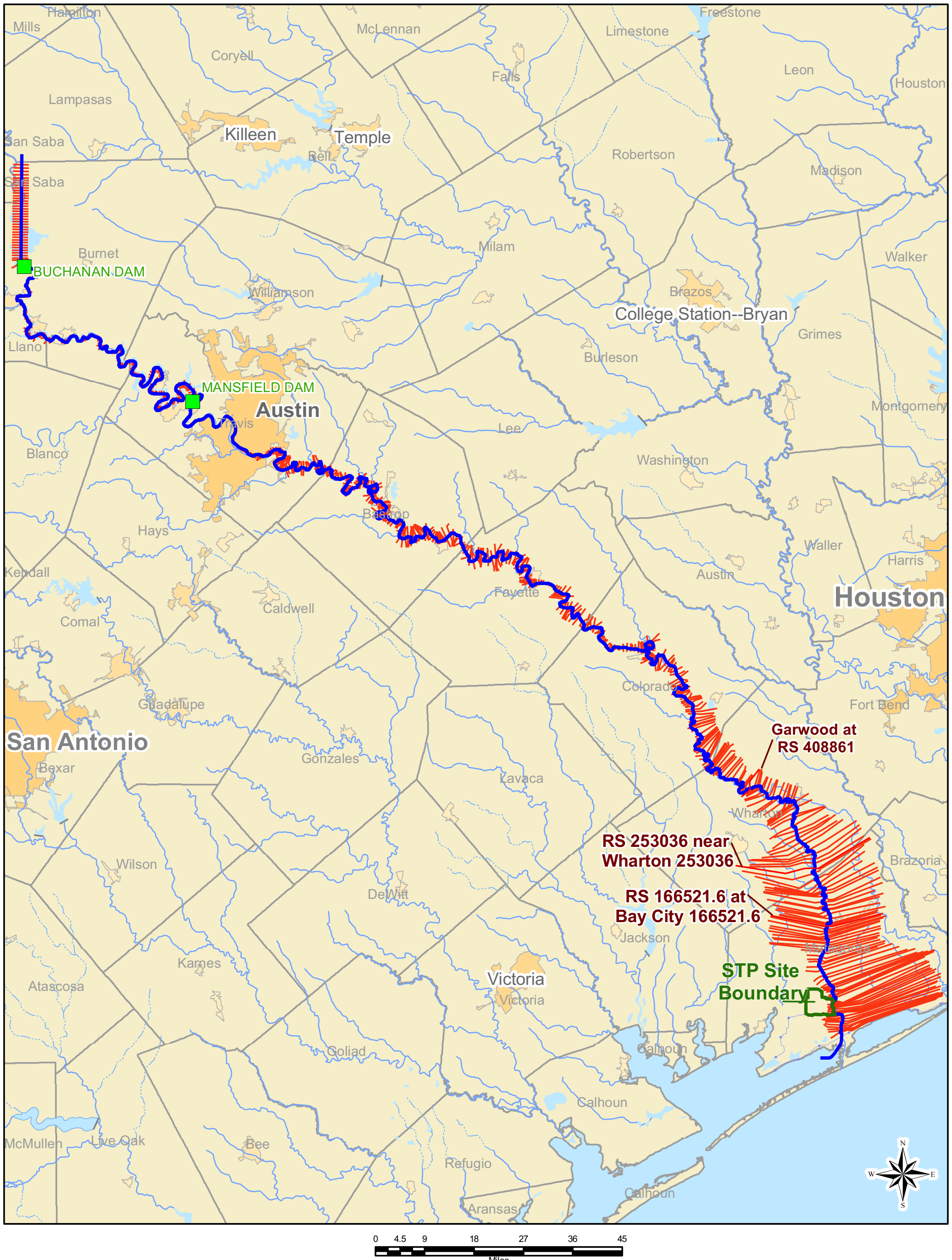


Figure 2.4S.4-4 Locations of Model Cross Sections in the Dam Break Analysis

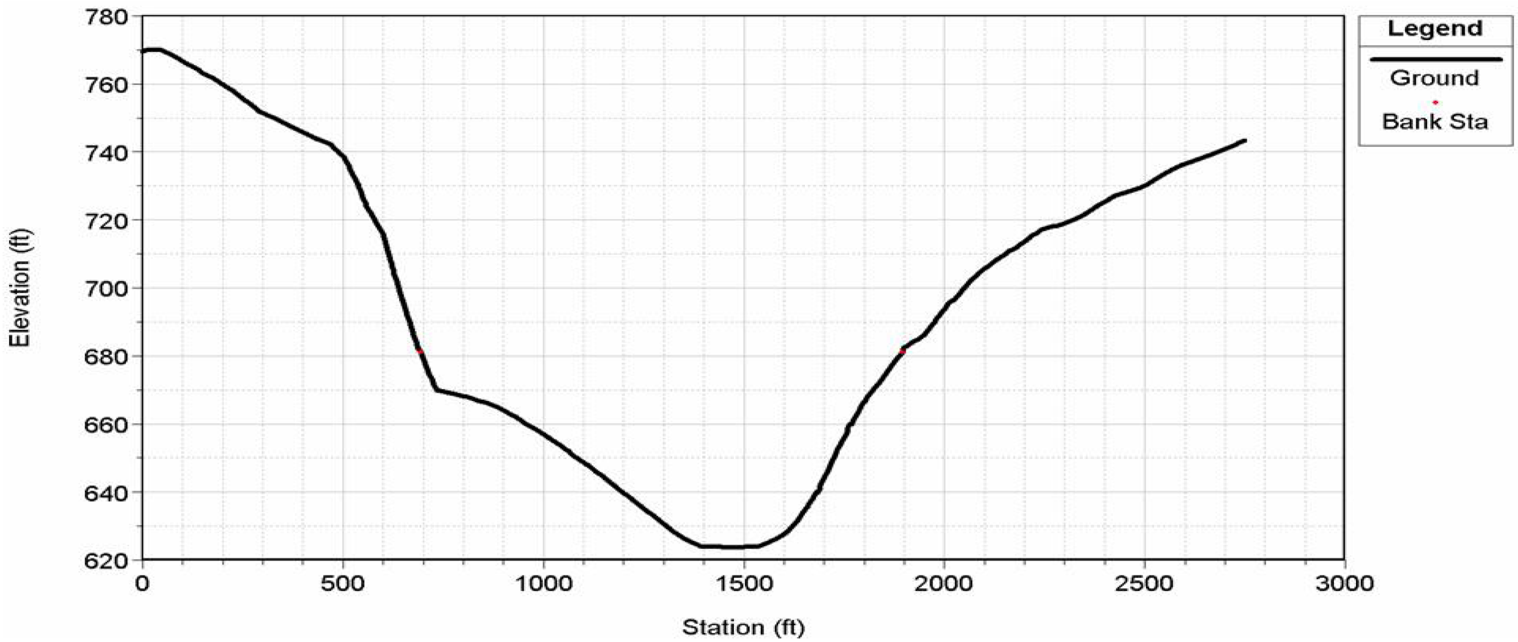


Figure 2.4S.4-5 Model River Cross Section at About 365 River Miles Upstream of the GIWW

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

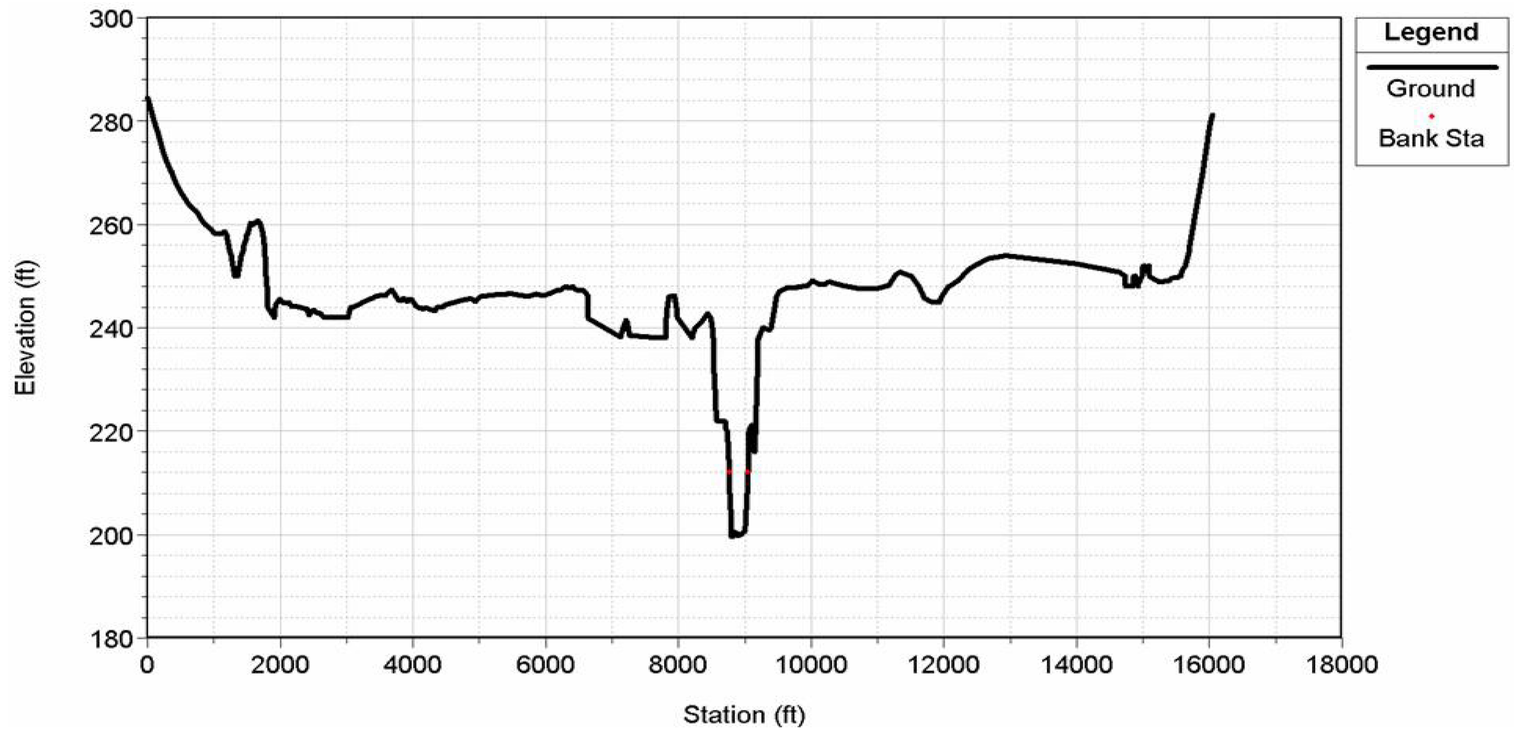


Figure 2.4S.4-6 Model River Cross Section at About 163.5 River Miles Upstream of the GIWW

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

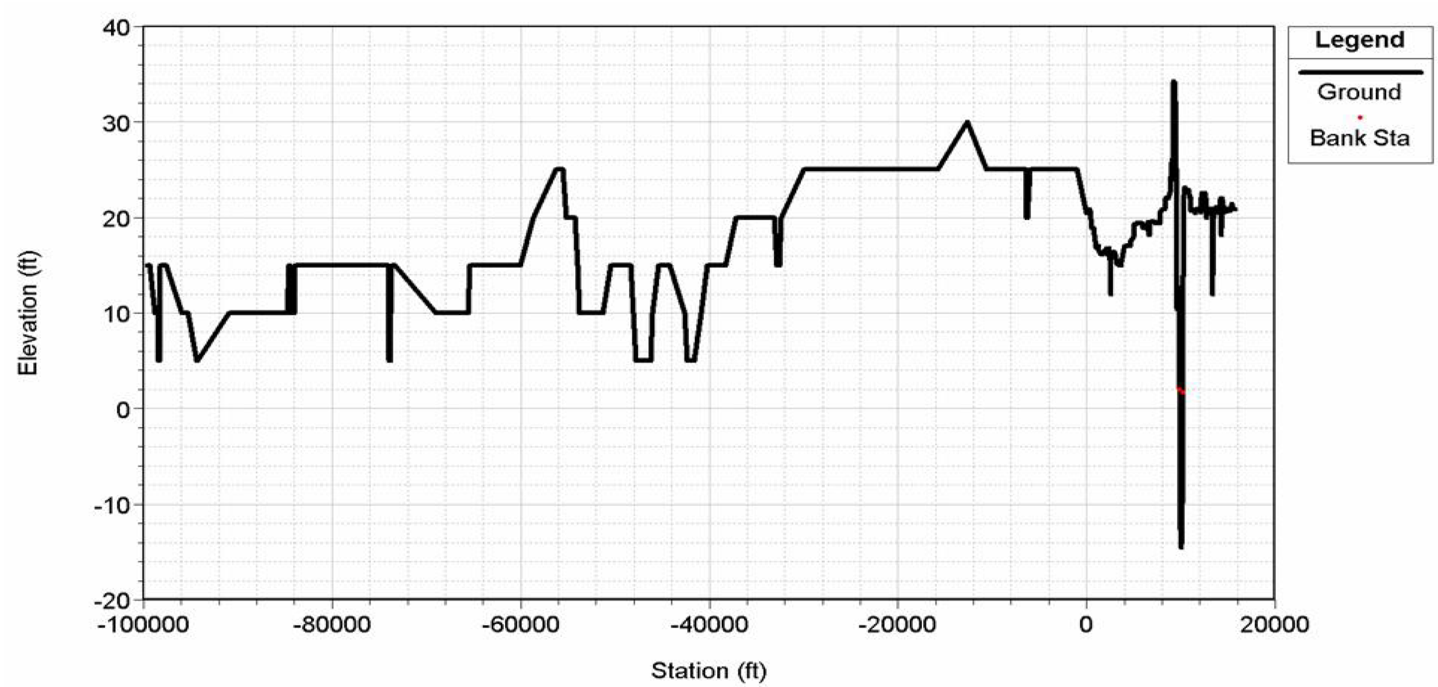


Figure 2.4S.4-7 Model River Cross Section at About 10.5 River Miles Upstream of the GIWW

Note: Near the STP site.

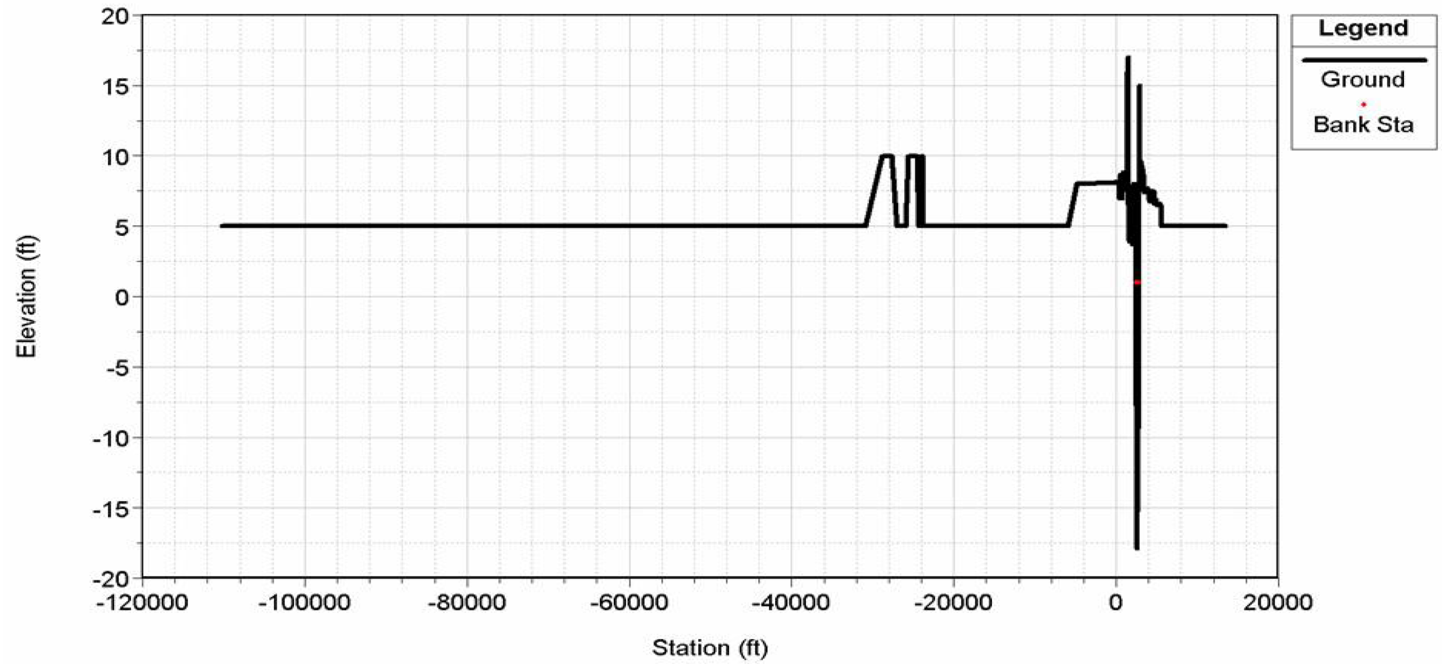


Figure 2.4S.4-8 Model River Cross Section at Downstream Model Boundary at about 0.9 River Miles Upstream of the GIWW

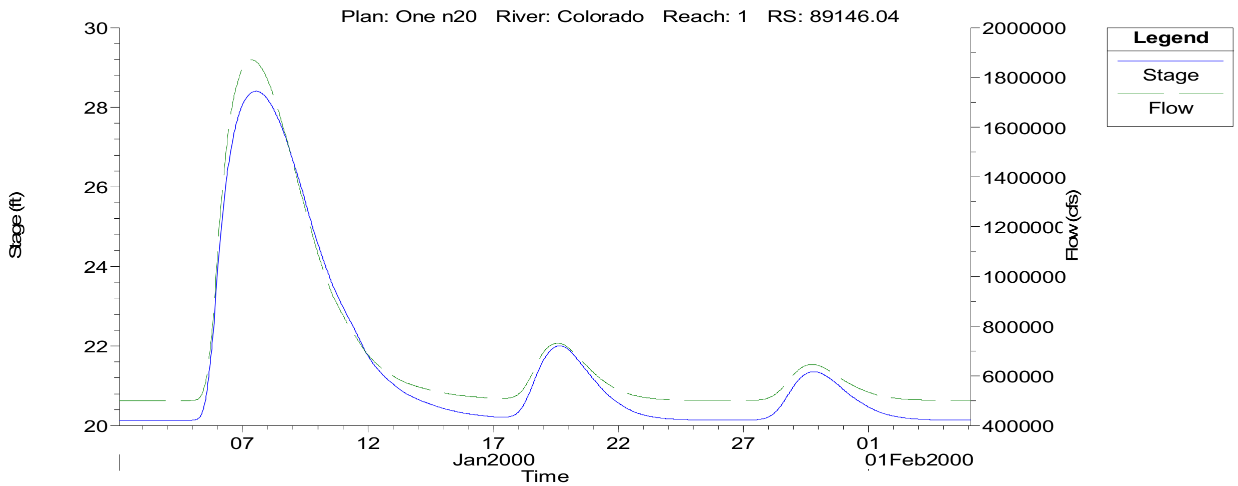


Figure 2.4S.4-9 Based Case Flood and Stage Hydrographs at the STP 3 & 4 Site

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

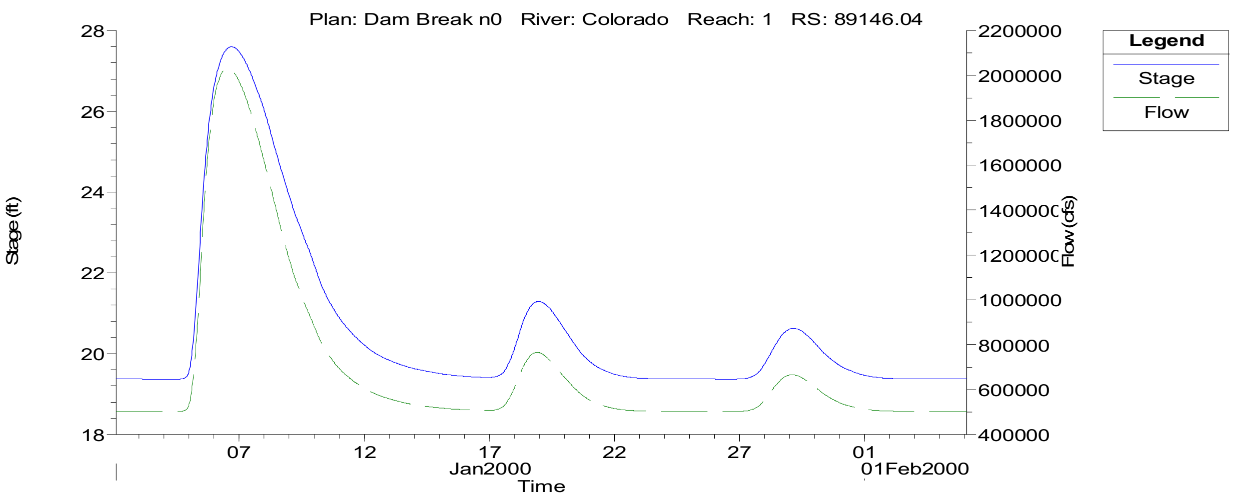


Figure 2.4S.4-10 Sensitivity Case Flood and Stage Hydrographs at the STP 3 & 4 Site

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

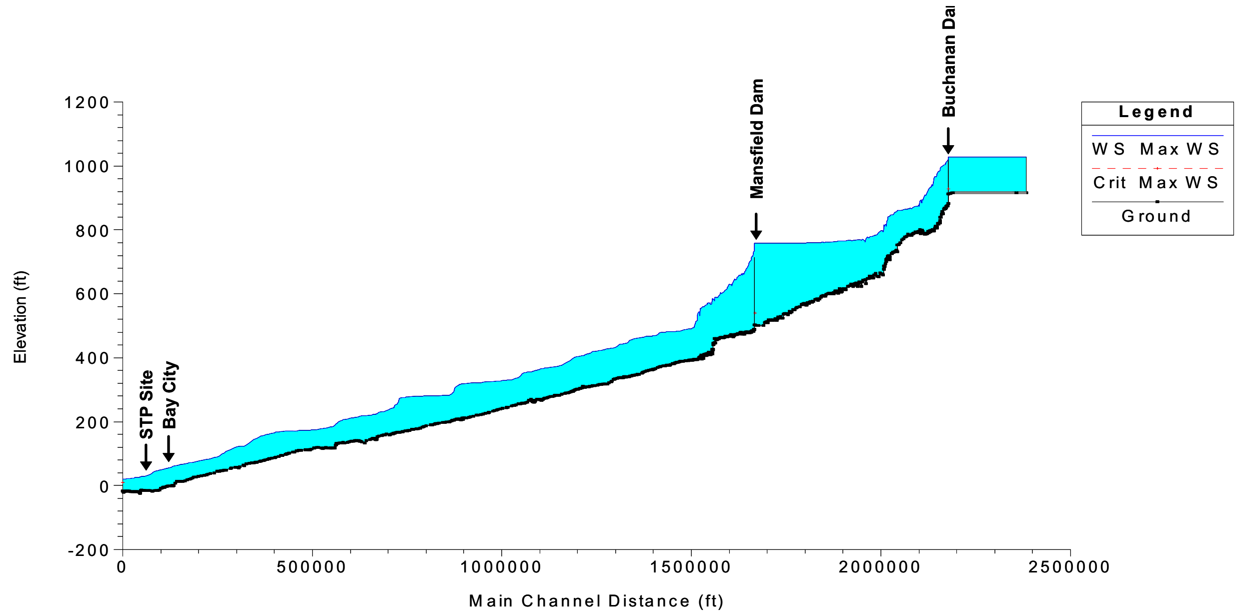


Figure 2.4S.4-11 Base Case Simulated Maximum Dam Break Surface Profiles from Buchanan Dam to 4,600 ft upstream of GIWW (Vertical Datum in NAVD 88)

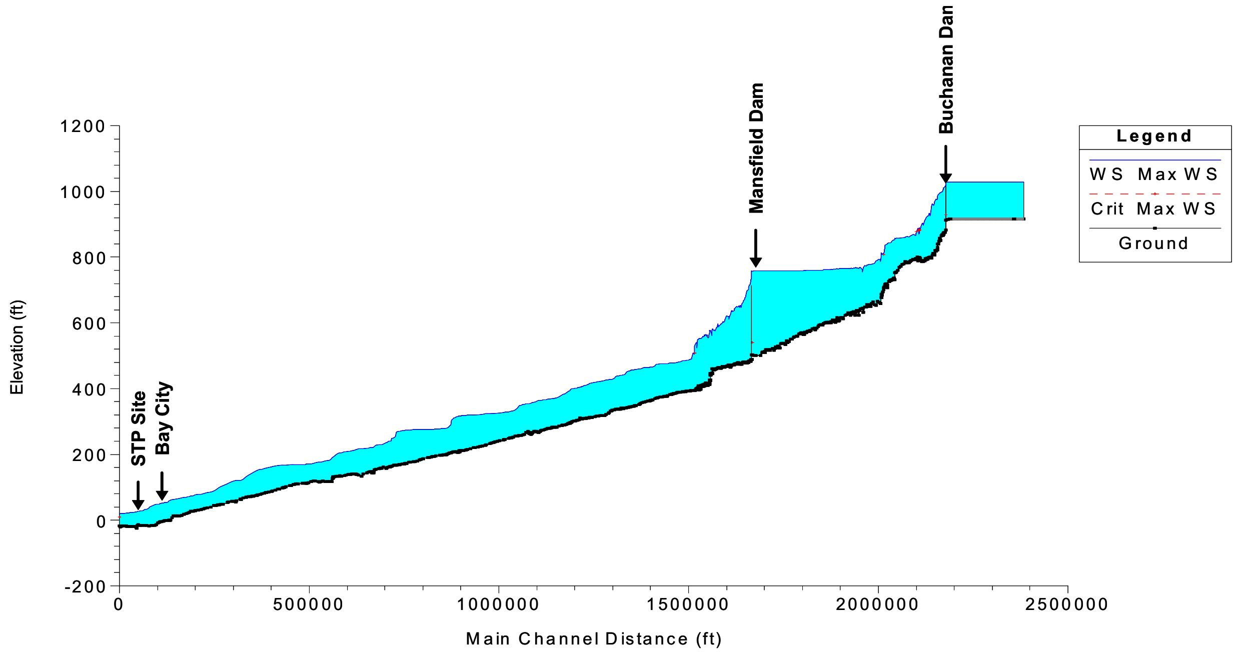


Figure 2.4S.4-12 Sensitivity Case Simulated Maximum Dam Break Surface Profiles from Buchanan Dam to 4600 ft Upstream of GIWW

Note: Vertical Datum in NAVD 88.

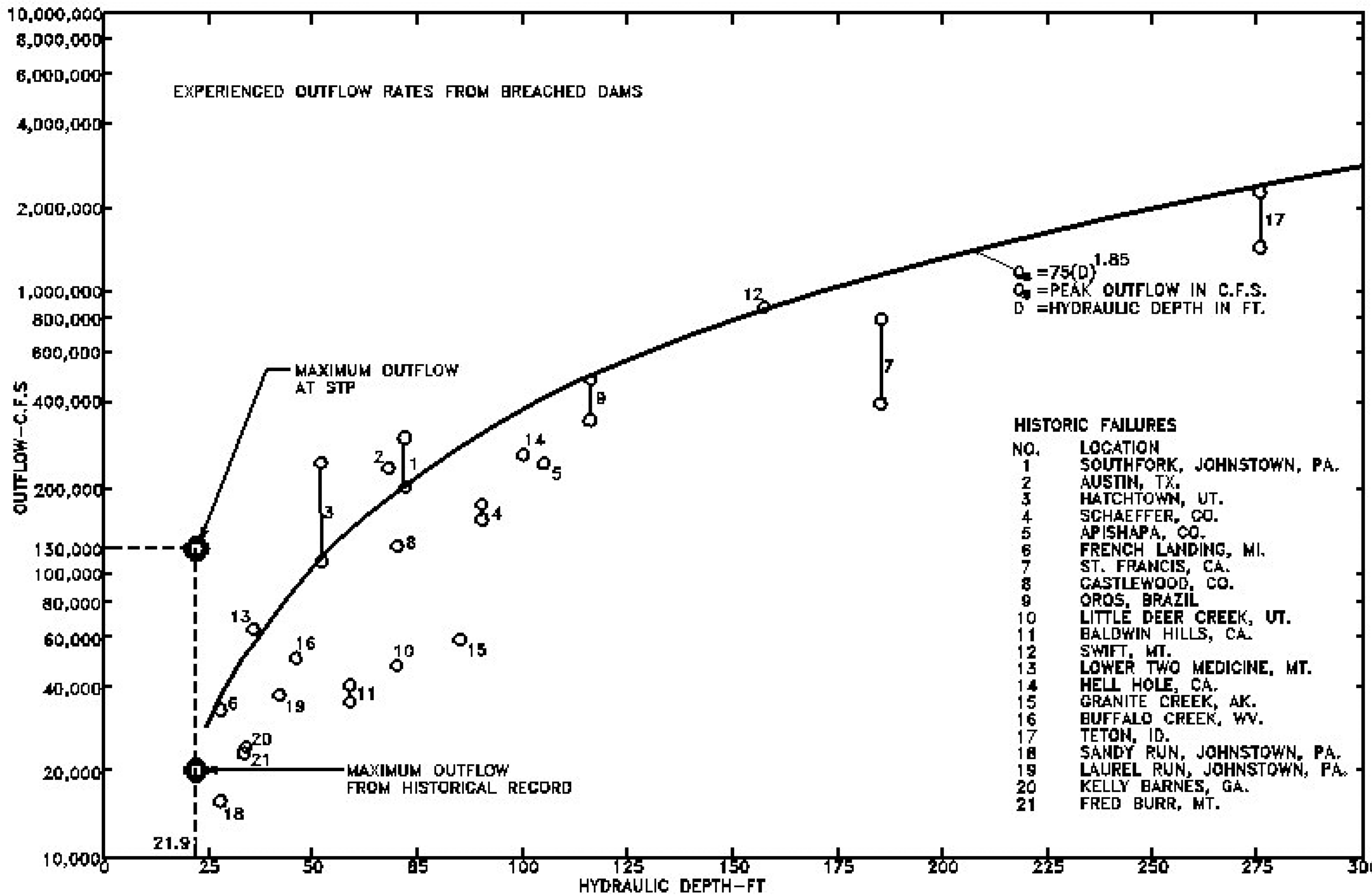


Figure 2.4S.4-13 Outflow Rates Experienced from Breached Dams (Reference 2.4S.4-12e)

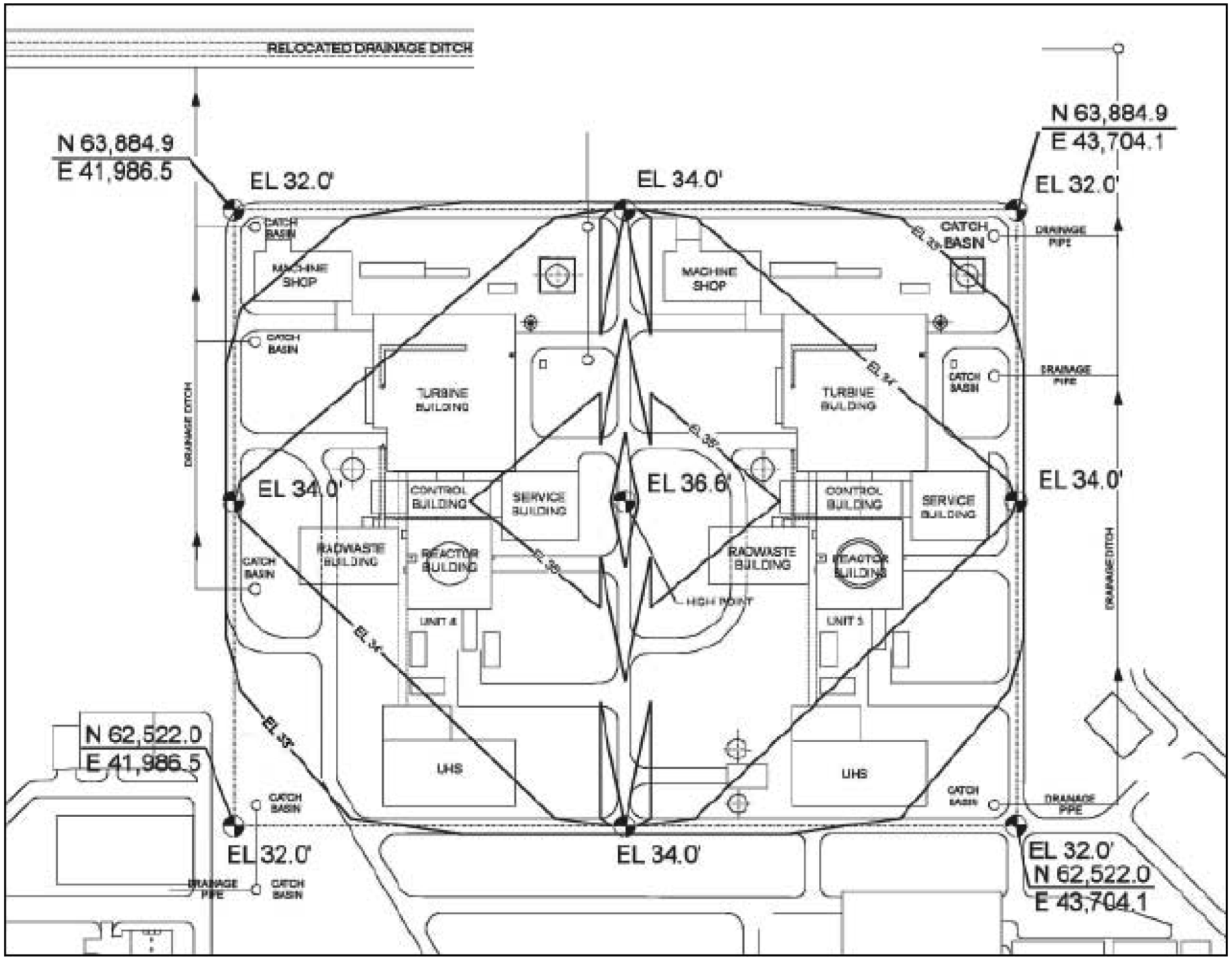


Figure 2.4S.4-14 Units 3 and 4 Site Grading Plan

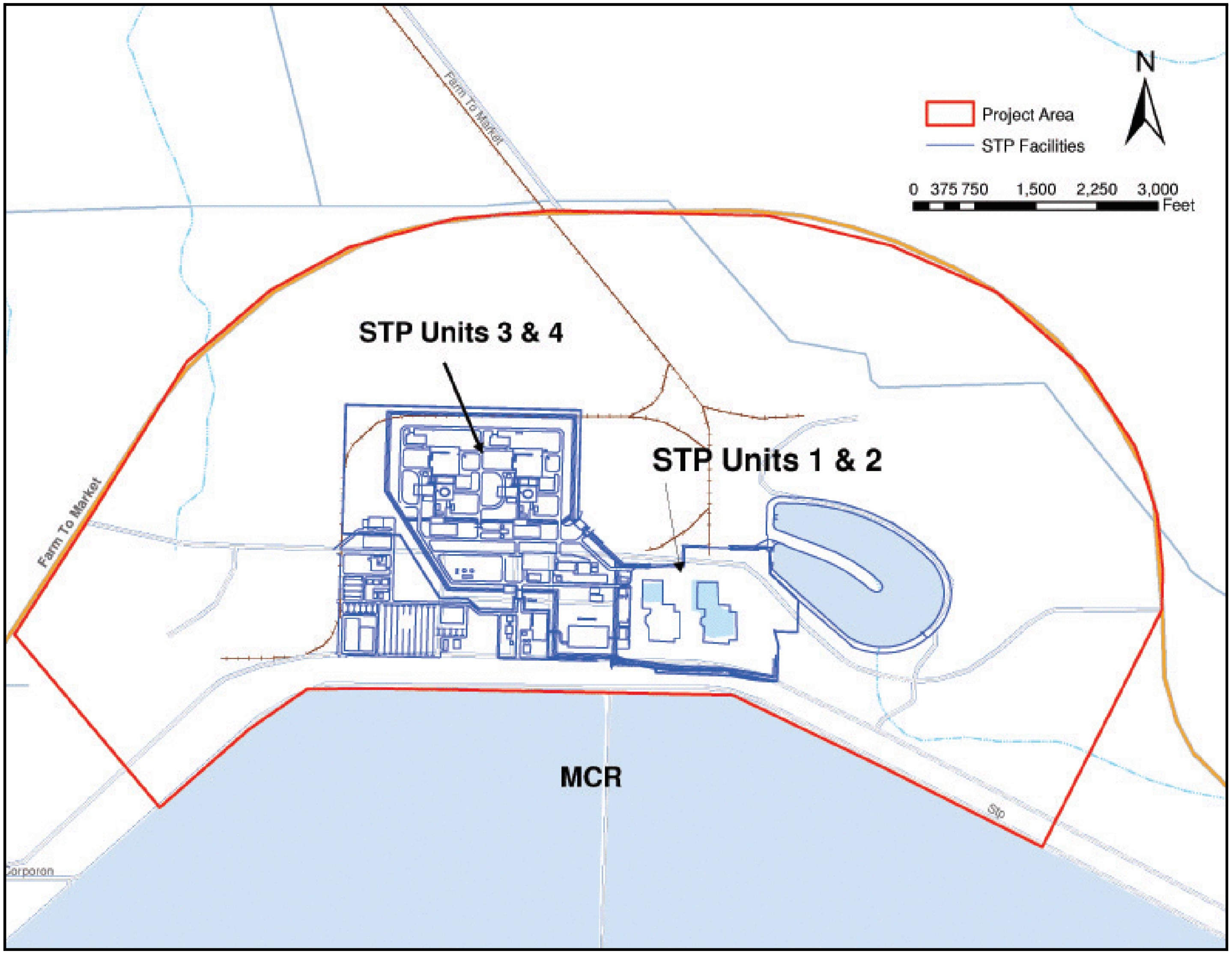


Figure 2.4S.4-15 STP Site Layout

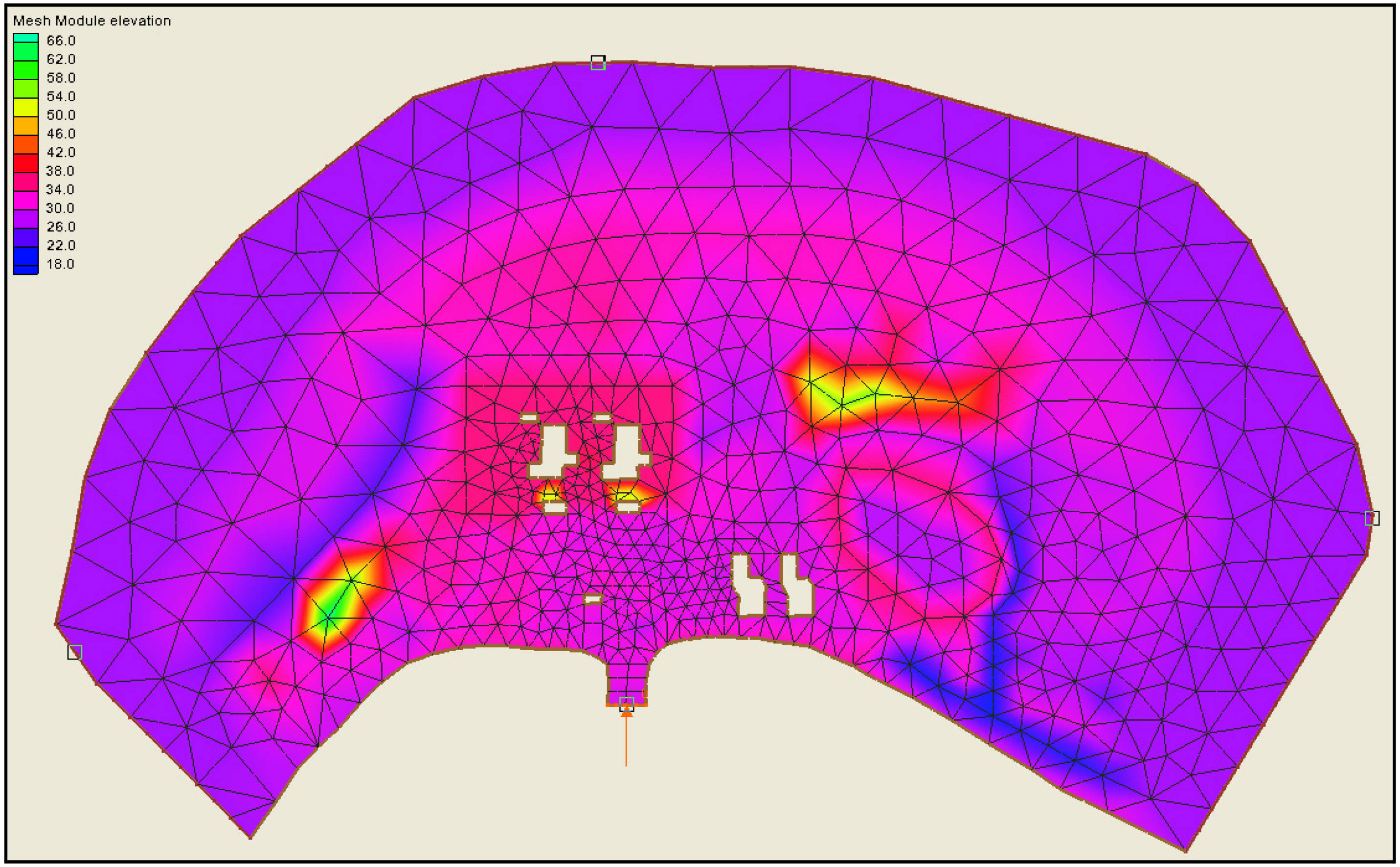


Figure 2.4S.4-16 Two-Dimensional View of Developed 2-D Grid with an East Breach

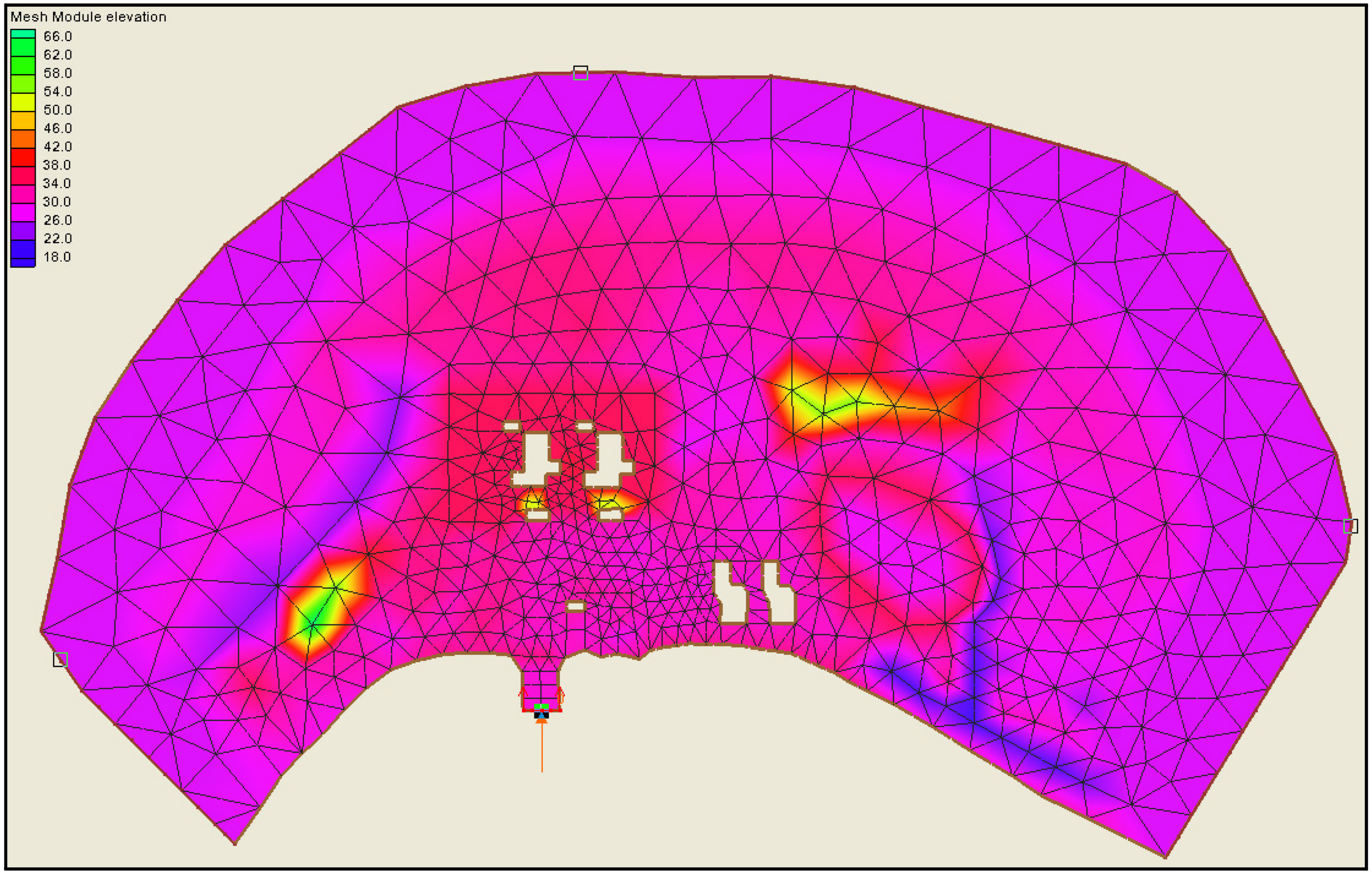


Figure 2.4S.4-17 Two-Dimensional View of Developed 2-D Grid with a West Breach

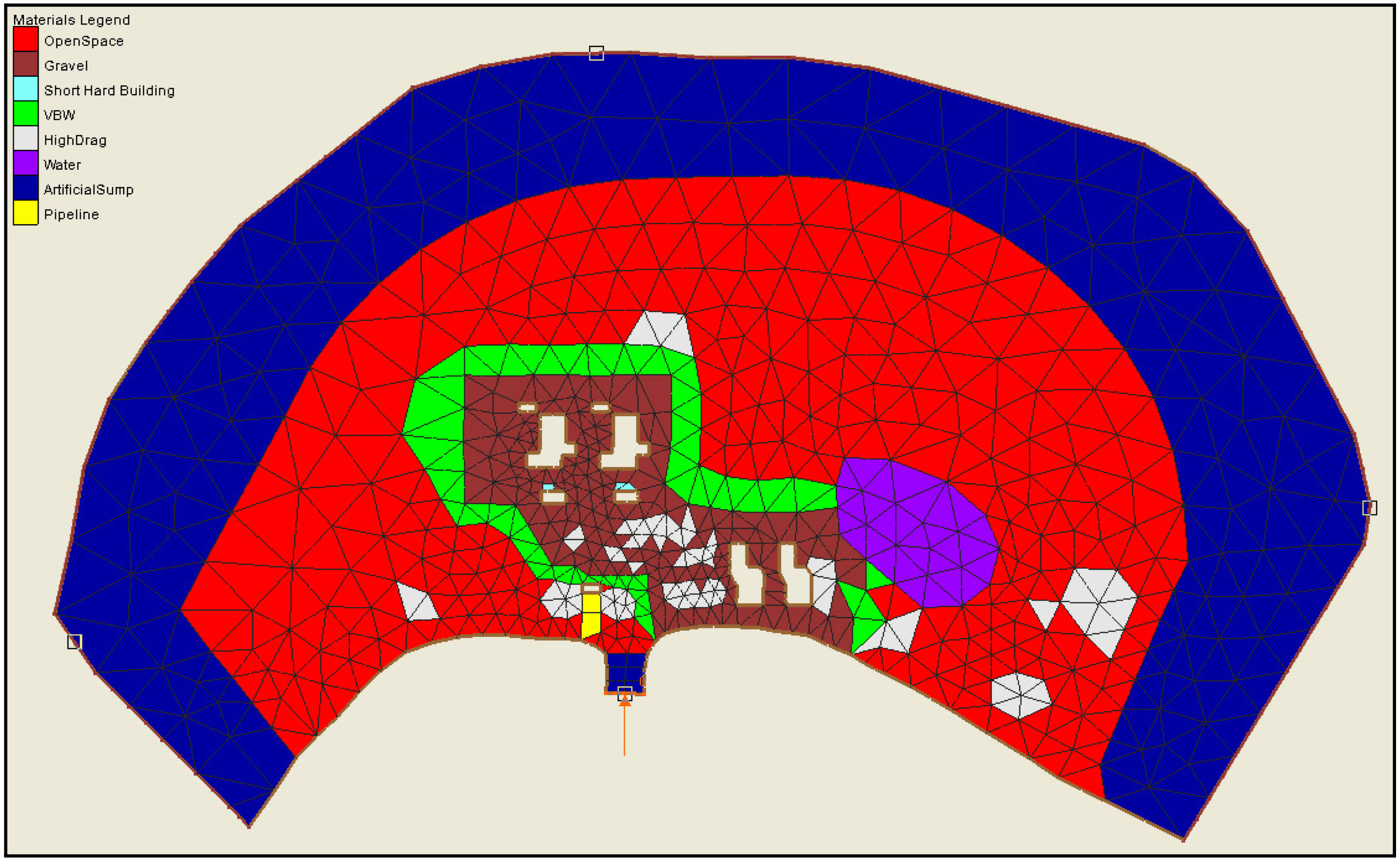


Figure 2.4S.4-18 Assigned Material Types of Developed 2-D Grid

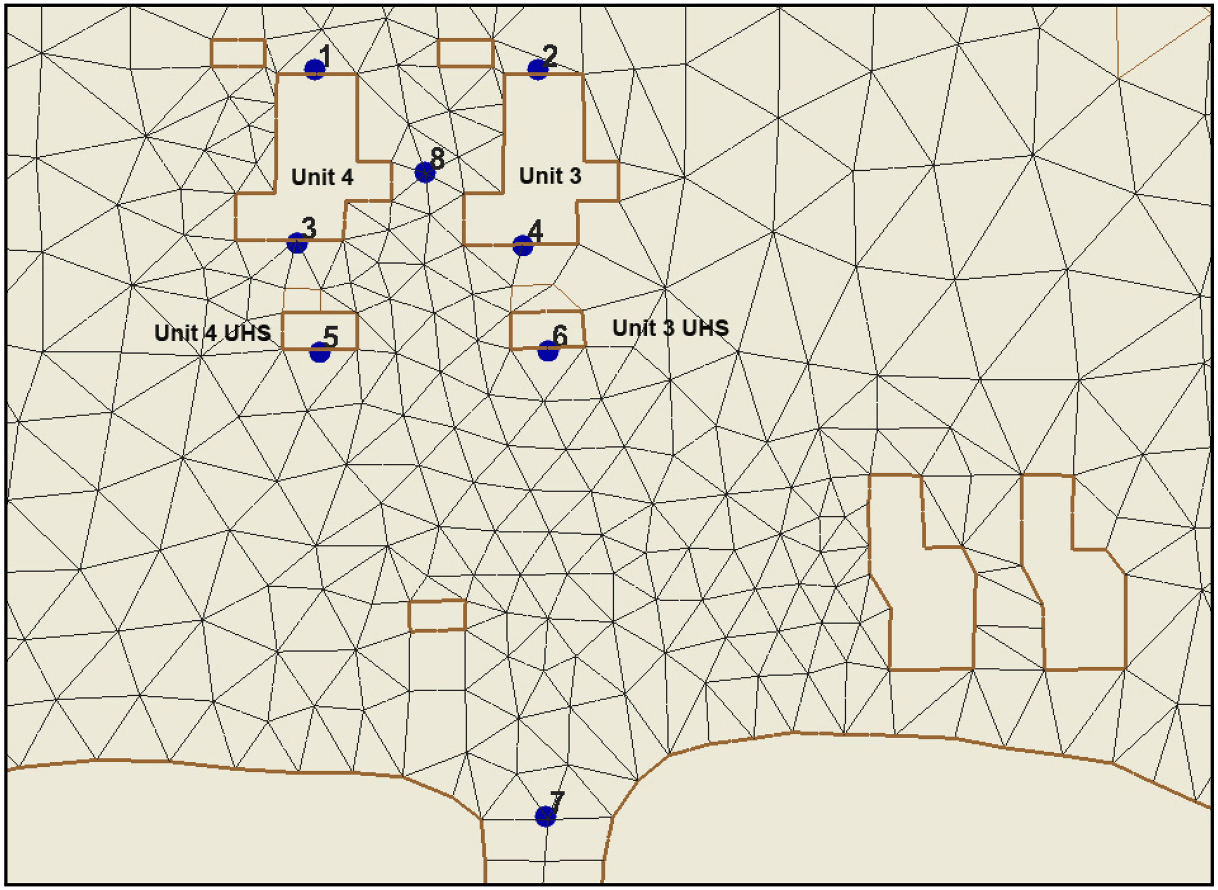


Figure 2.4S.4-19 Locations for RMA2 Modeling Results

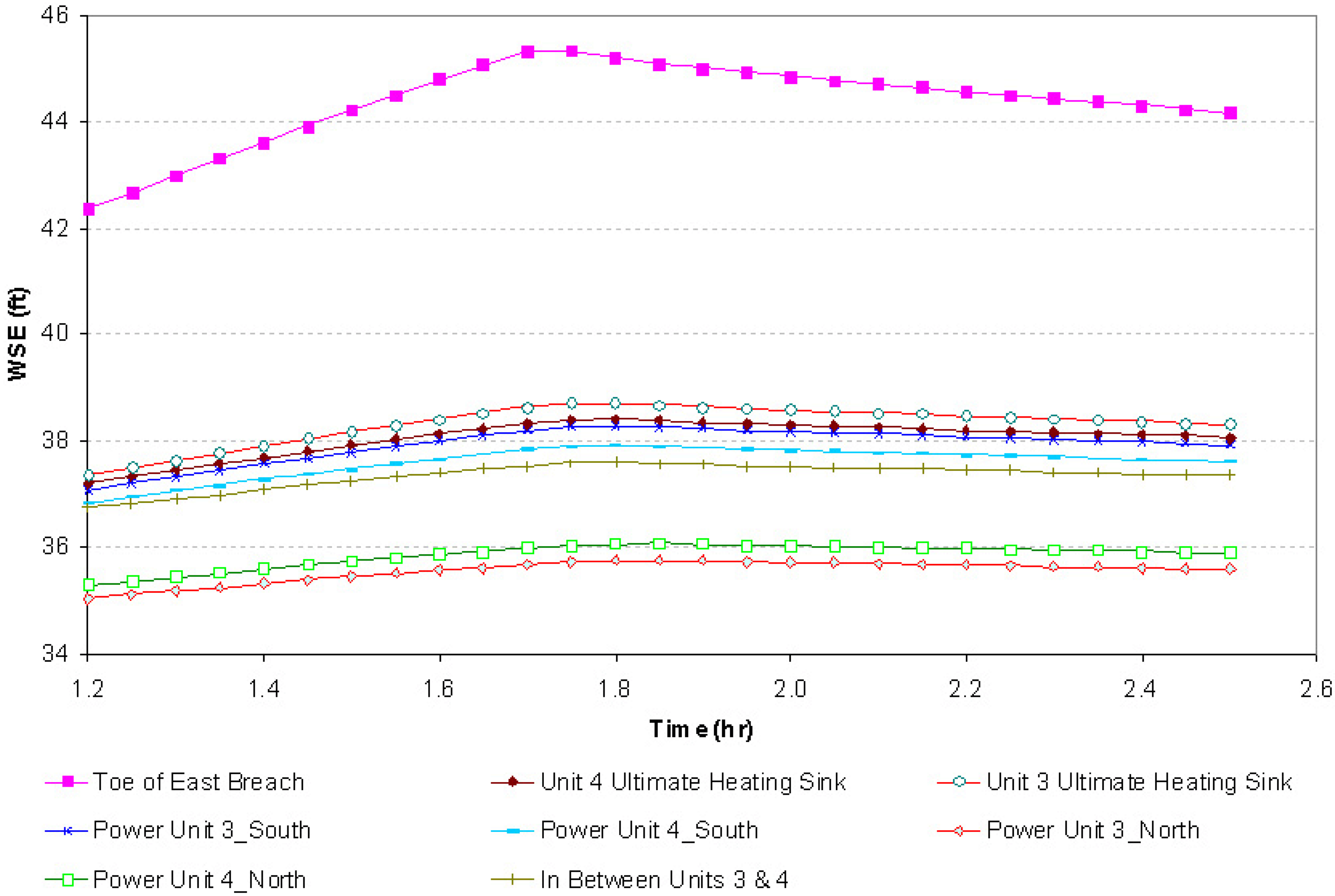


Figure 2.4S.4-20 Time-Dependent Water Surface Elevations Associated with East Breach Scenario

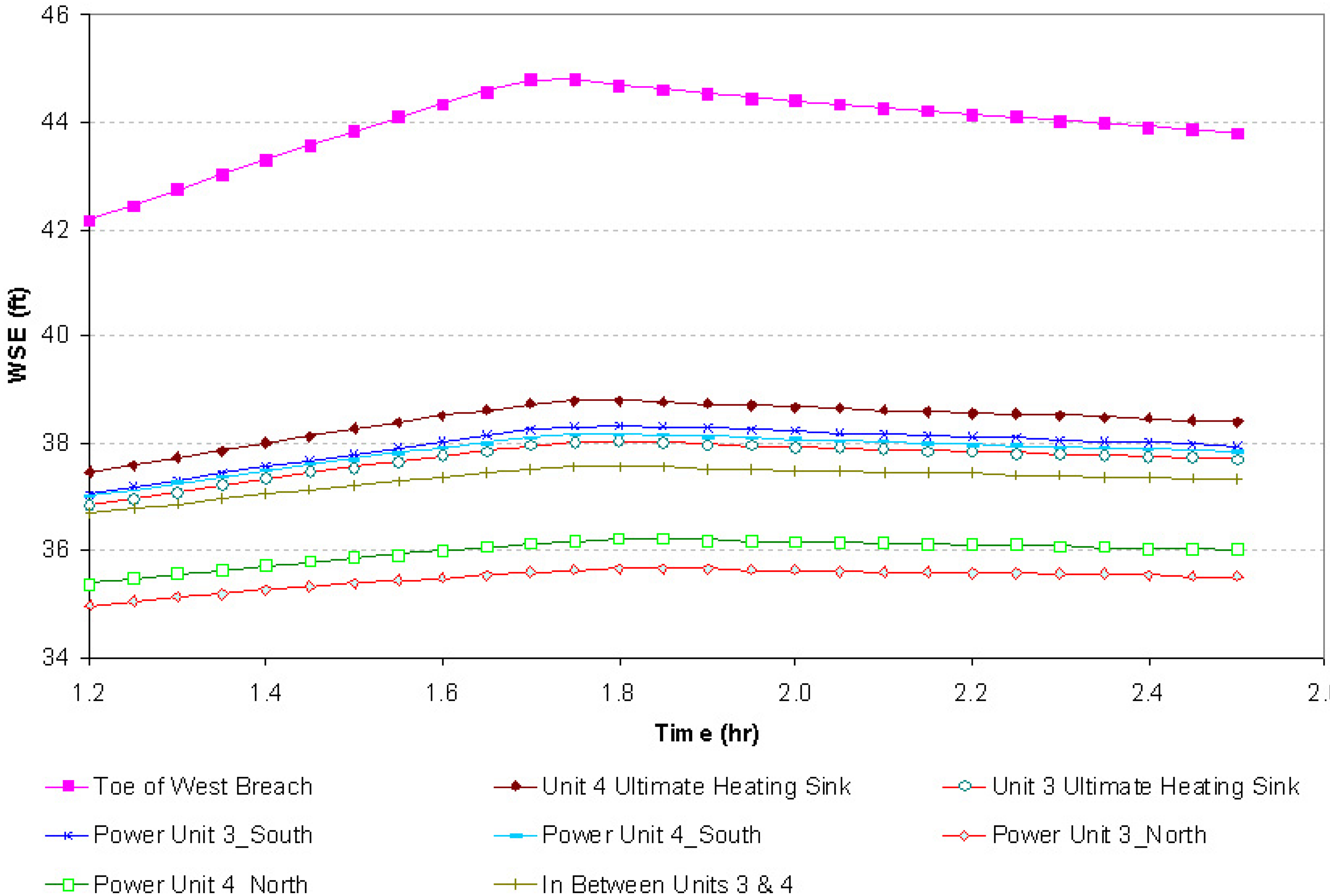


Figure 2.4S.4-21 Time-Dependent Water Surface Elevations Associated with West Breach Scenario

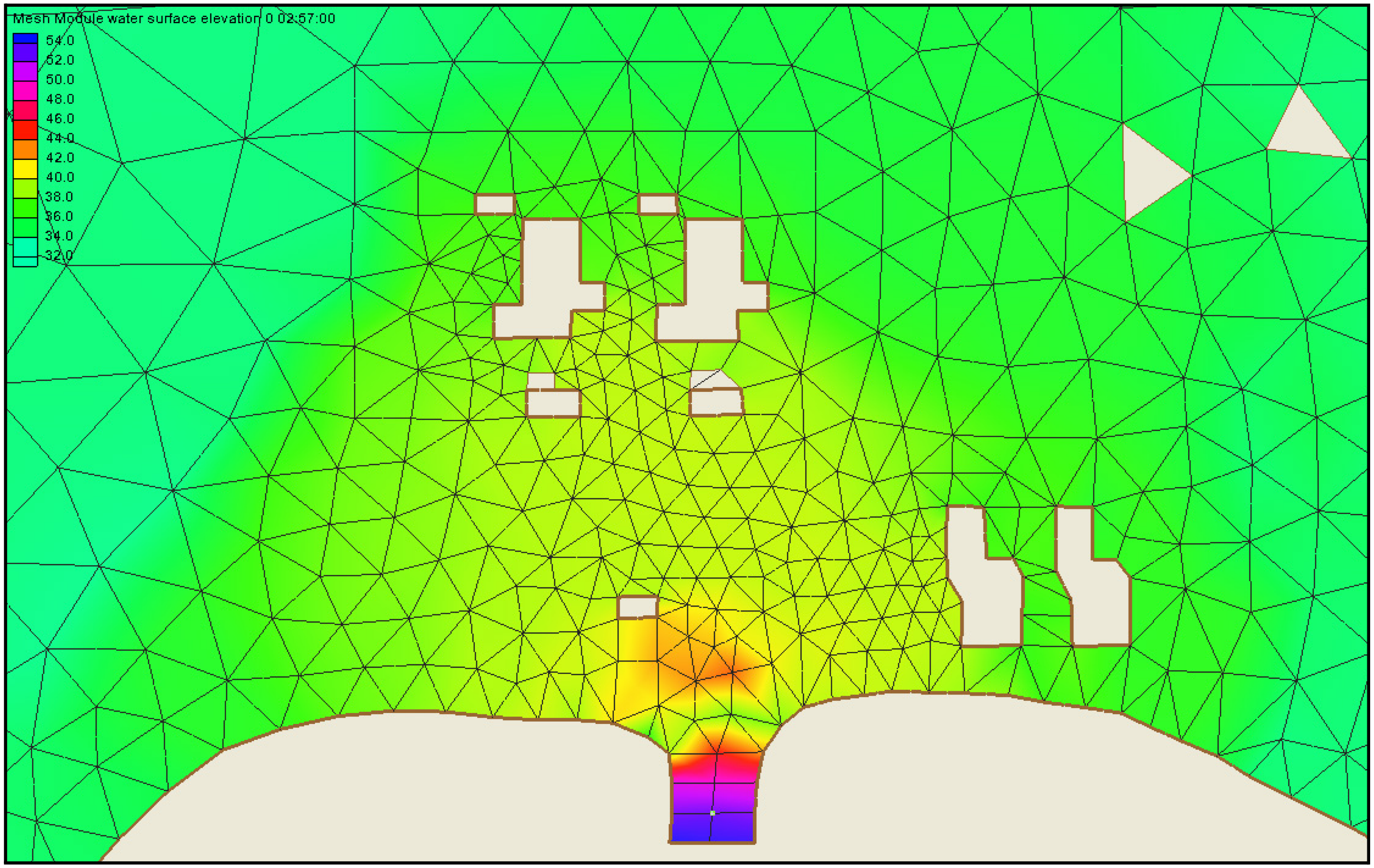


Figure 2.4S.4-21a Peak Water Surface Elevations Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)

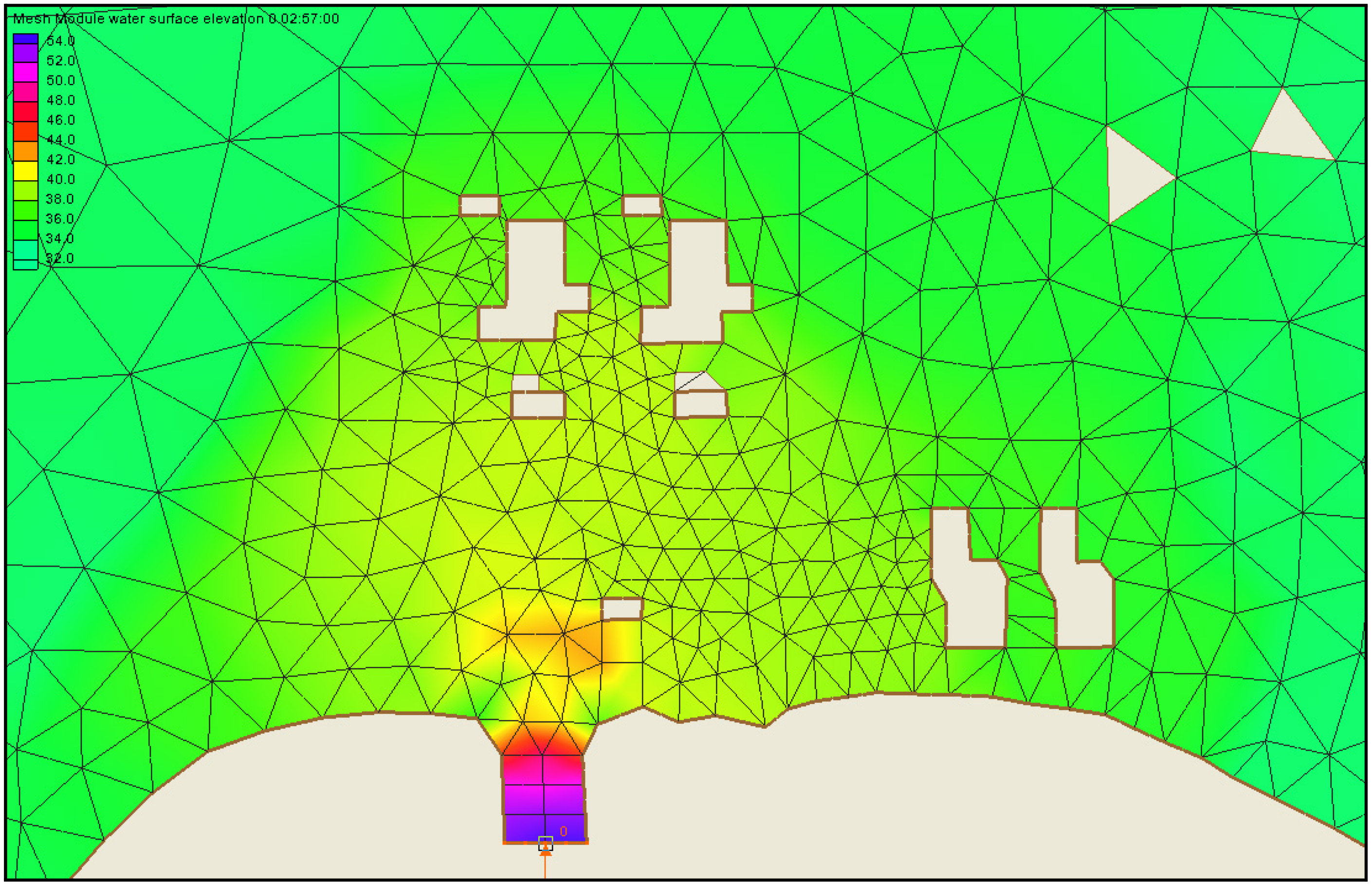


Figure 2.4S.4-21b Peak Water Surface Elevations Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)

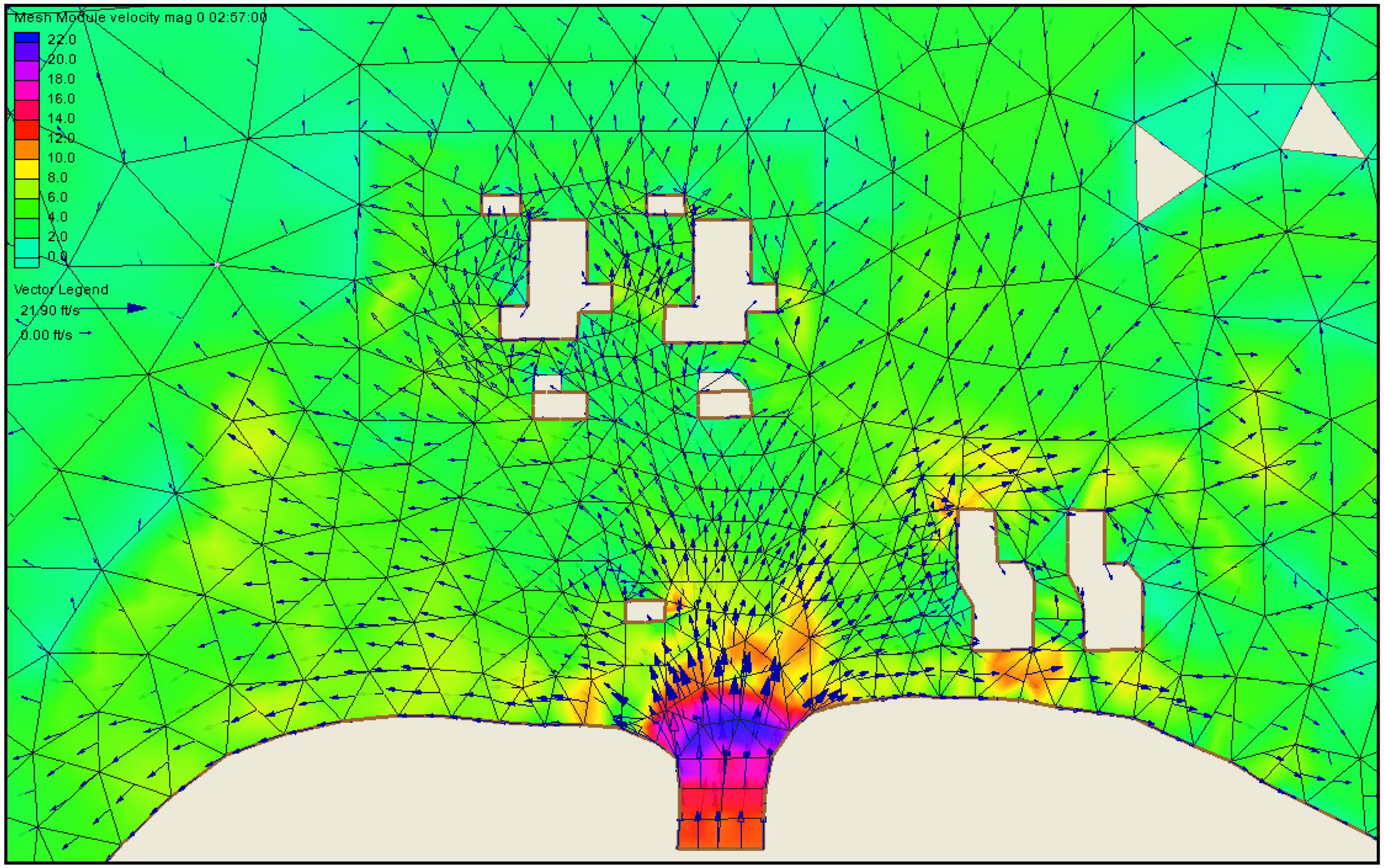


Figure 2.4S.4-21c Peak Velocities Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)

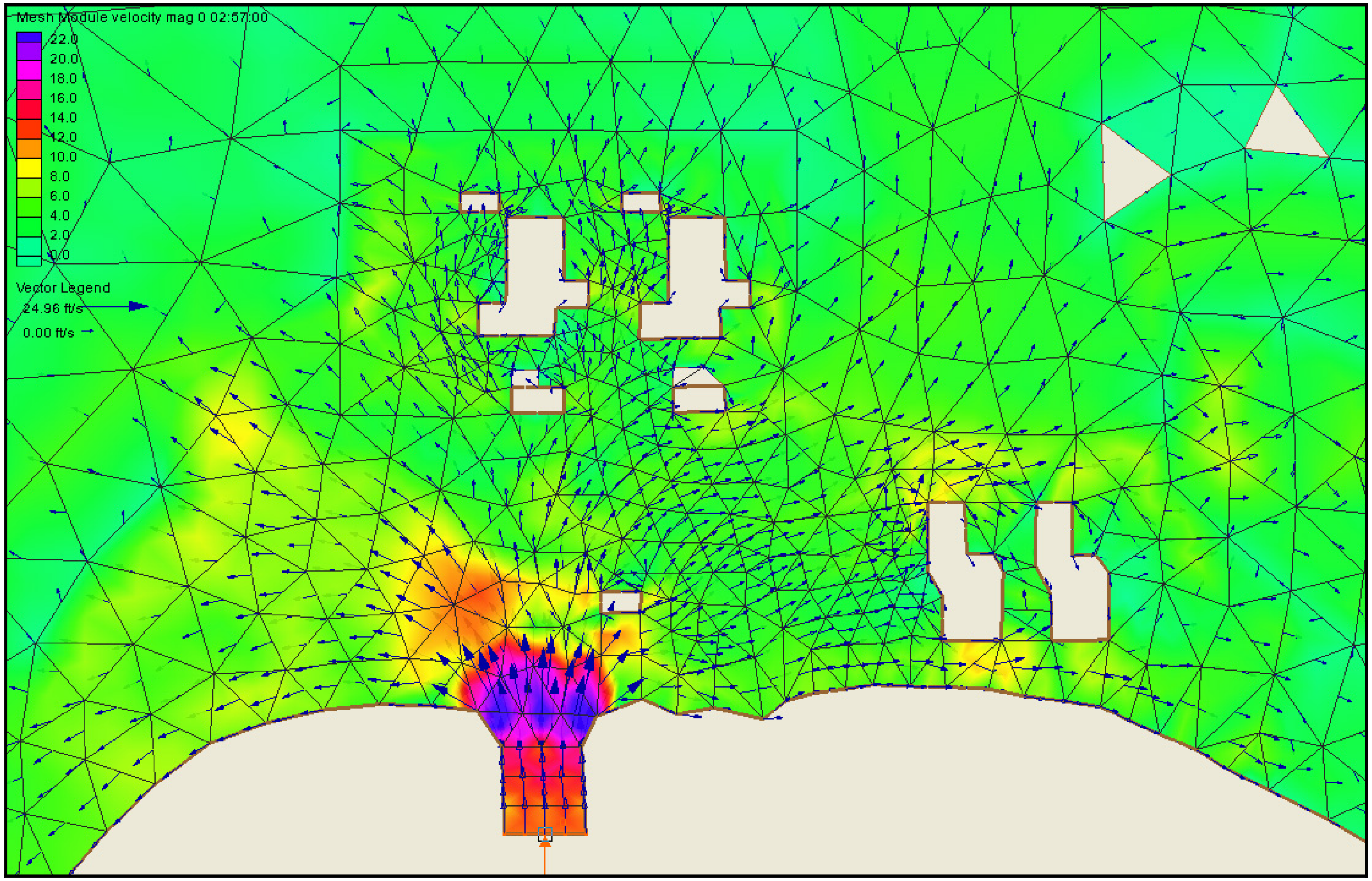


Figure 2.4S.4-21d Peak Velocities Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)

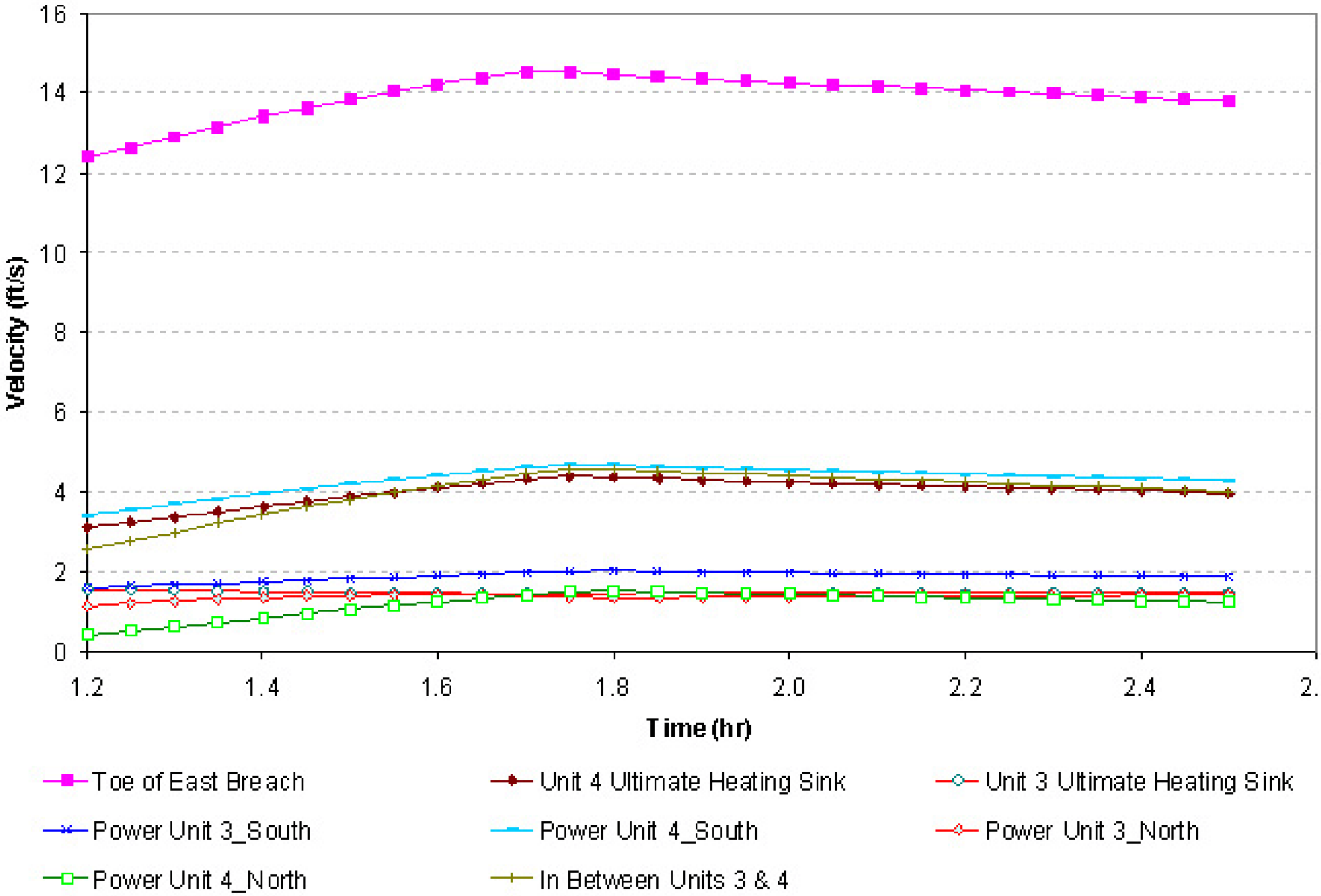


Figure 2.4S.4-21e Time-Dependent Velocities Associated with East Breach Scenario

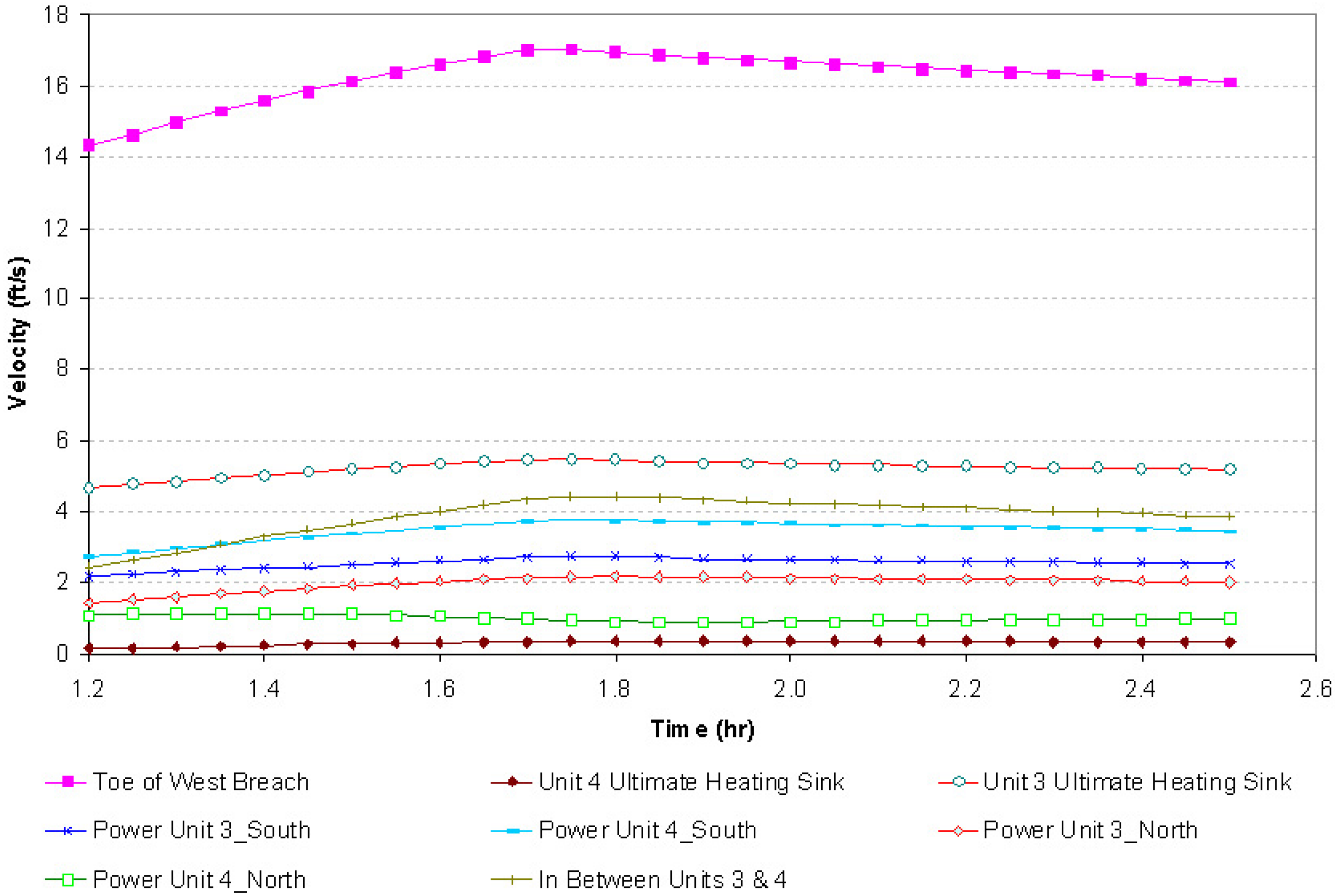


Figure 2.4S.4-21f Time-Dependent Velocities Associated with West Breach Scenario

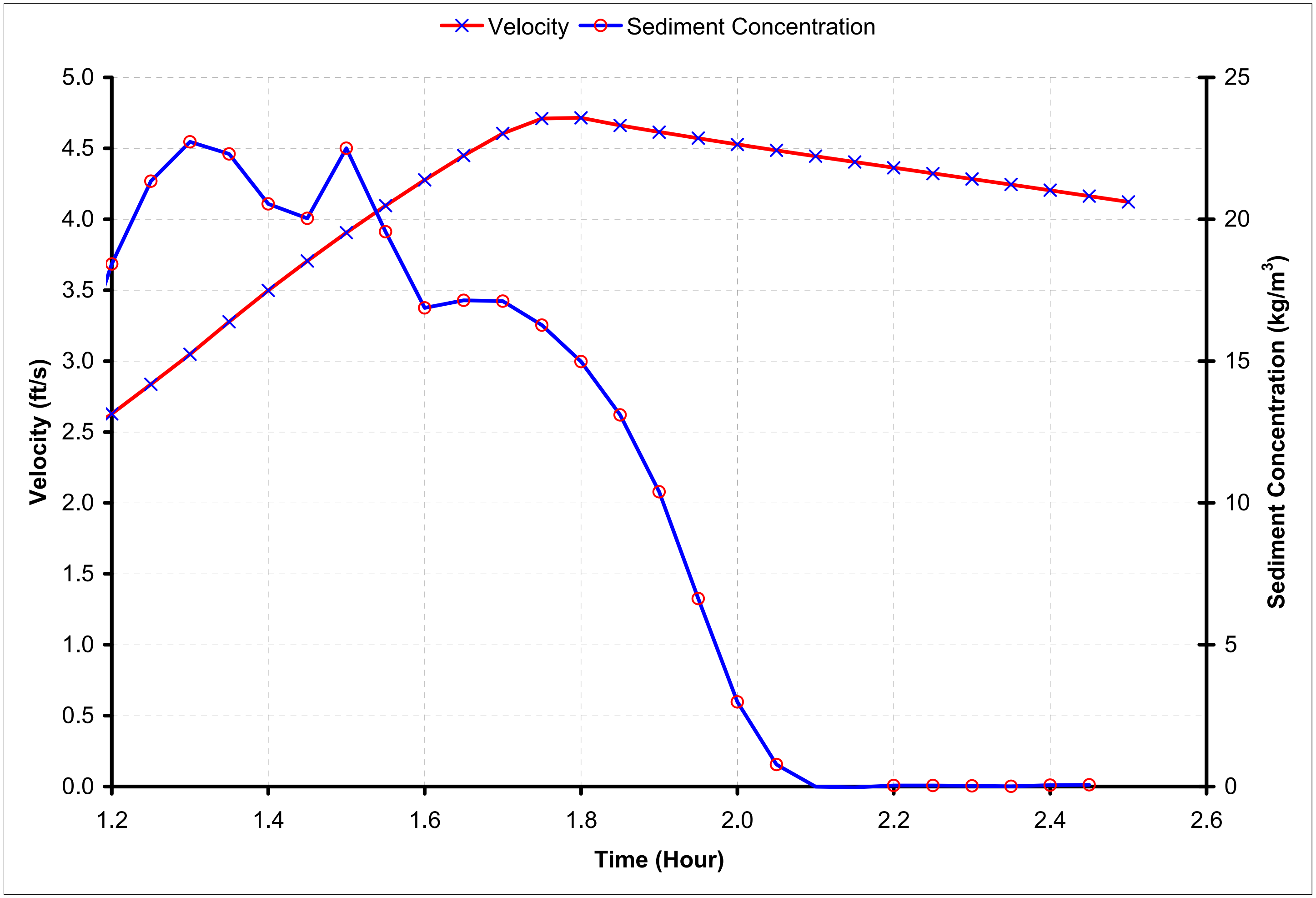


Figure 2.4S.4-21g Velocities and Sediment Concentrations In Between Units 3 and 4 with East Breach Scenario

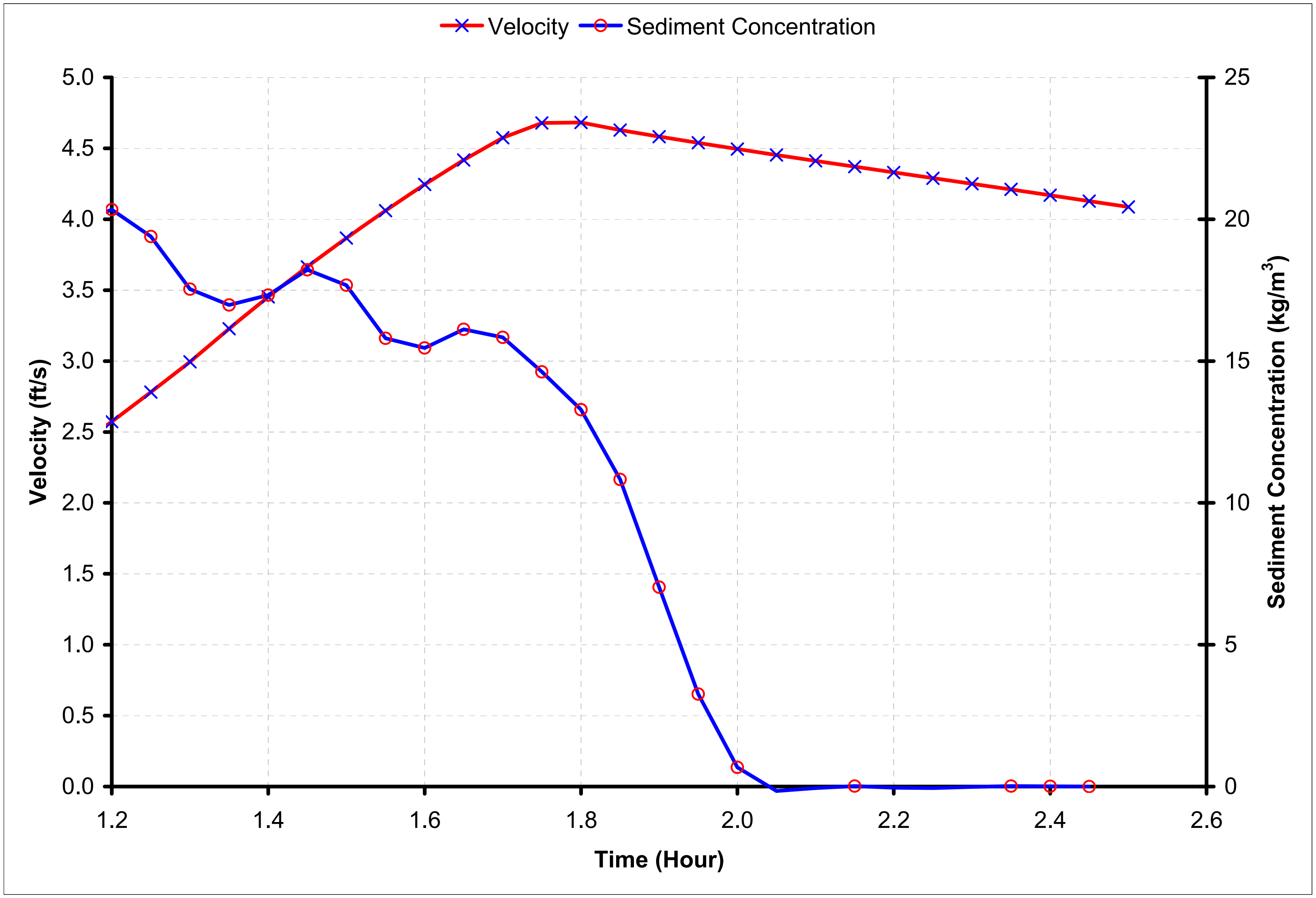


Figure 2.4S.4-21h Velocities and Sediment Concentrations In Between Units 3 and 4 with West Breach Scenario

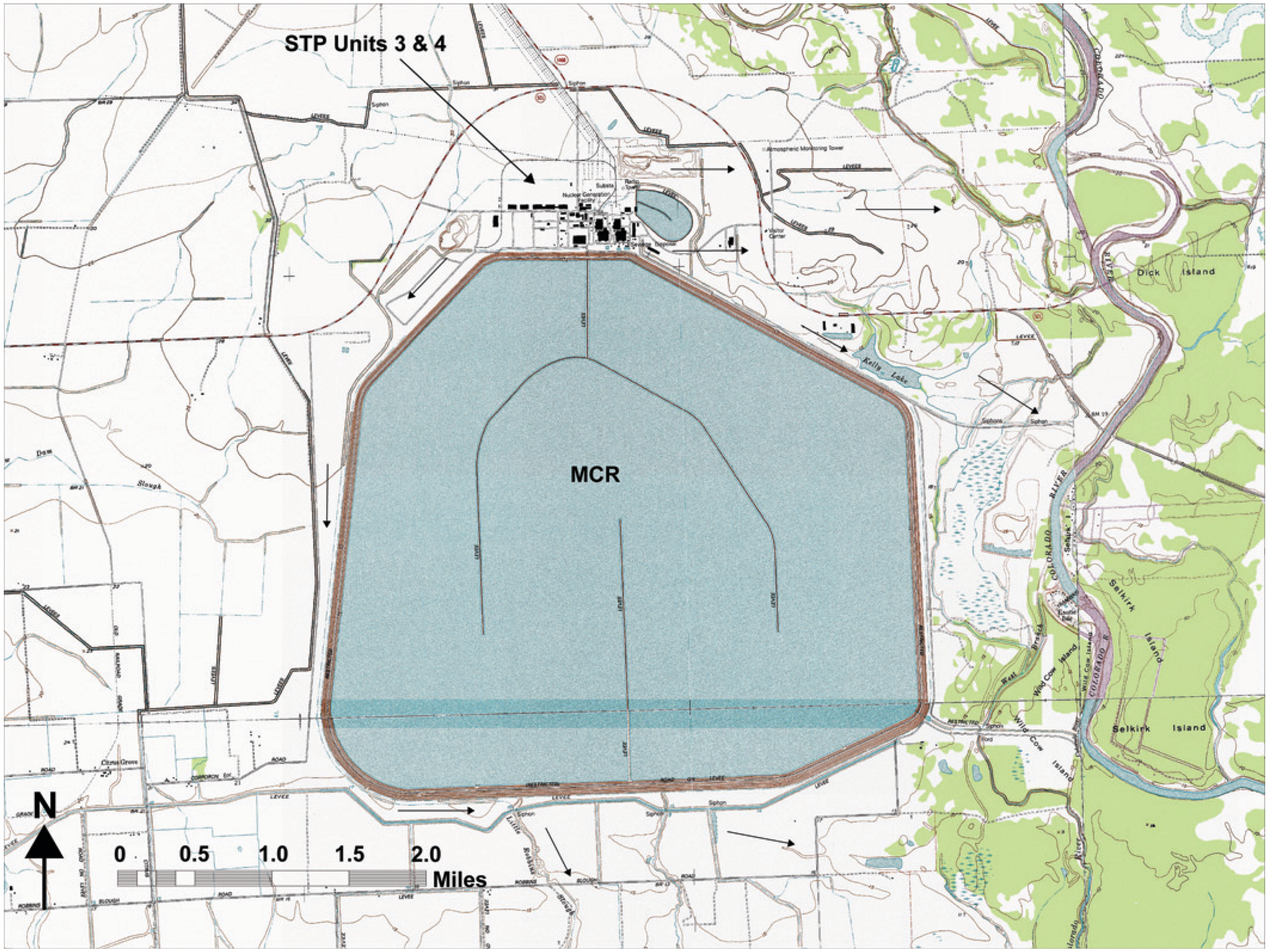


Figure 2.4S.4-21i Stream System around STP Site

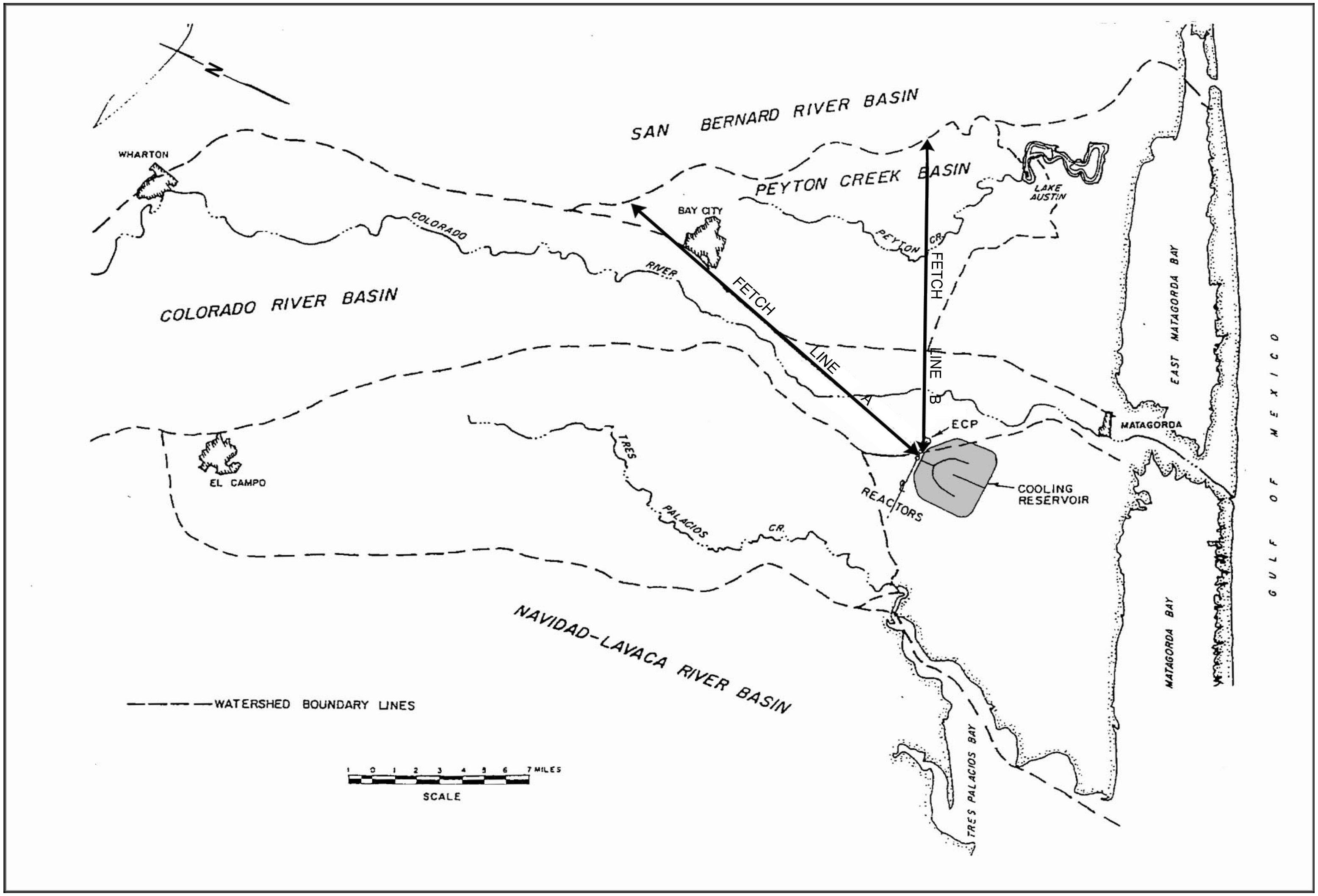


Figure 2.4S.4-22 Fetch Directions and Length