2.4S.3 Probable Maximum Flood (PMF) on Streams and Rivers

The following site-specific supplement addresses COL License Information Item 2.15.

The STP 3 & 4 site is located near the west bank of Lower Colorado River in Matagorda County, Texas, as shown in Figure 2.4S.3-1. The site is 12 miles south-southwest of Bay City, Texas and 8 miles north-northwest of Matagorda, Texas. There are a total of 68 dams with storage capacity in excess of 5000 acre-ft upstream of the STP 3 & 4 site on the Colorado River and its tributaries, as discussed in Subsection 2.4S.4. The Lower Colorado River Authority (LCRA) operates six of these dams that together form the six Highland Lakes: Buchanan, Inks, LBJ (with Wirtz Dam), Marble Falls (with Starcke Dam), Travis (with Mansfield Dam), and Austin (Tom Miller Dam), as discussed in Subsection 2.4S.1. The Highland Lake System was designed for flood management, water supply management, and hydroelectricity generation purposes. These lakes on the Colorado River are shown in Figure 2.4S.1-6.

In this subsection the Probable Maximum Flood (PMF) in the Lower Colorado River is analyzed to assess the flooding potential on the safety-related facilities at the STP 3 & 4 site. Several publicly available flood hydrologic studies performed on the Lower Colorado River basin from 1985 to 2002 (References 2.4S.3-1 to 2.4S.3-8) by Federal, State, and other local agencies were reviewed to establish the combination of events that constitute the probable maximum flood condition at the STP 3 & 4 site.

The probable maximum flood estimates provided in these studies are still applicable to the present hydrologic conditions in the Lower Colorado River basin because a) the major hydrologic features (including dams and reservoirs) in the river basin have not changed since1985 and b) the Probable Maximum Precipitation (PMP) estimates provided in these studies, which are used to estimate the PMF, are based on current hydrologic design procedures. The flood hydrologic studies reviewed are:

- Possible PMF scenarios considered for STP 1 & 2 and reported in the Updated Final Safety Analysis Report (UFSAR) (Reference 2.4S.3-1).
- PMF estimates and dam safety evaluation studies for Mansfield Dam by the United States Bureau of Reclamation (USBR) (References 2.4S.3-2, 2.4S.3-3, and 2.4S.3-4) and others (References 2.4S.3-5 and 2.4S.3-6).
- Dam safety evaluation project for the six Highland Lakes in the Lower Colorado River (from Lake O.H. Ivie to Mansfield Dam), Phase II (Preliminary Design and Final Design), by Freese & Nichols Inc. (Reference 2.4S.3-7).
- Flood damage evaluation project for the Lower Colorado River (from Lake O.H. Ivie to the Gulf of Mexico at Matagorda Bay) by Halff Associates Inc. (Reference 2.4S.3-8).

A brief overview of each of these studies is given in Subsection 2.4S.3.4.1.

The possible PMF scenarios considered for the existing STP 1 & 2 (Reference 2.4S.3-1) were evaluated for their applicability to the present and forecast future conditions of the Lower Colorado River based on information provided in the hydrologic studies reviewed (References 2.4S.3-2, 2.4S.3-7, and 2.4S.3-8) and in the Region “K” Plan of the 2007 State Water Plan adopted by the Texas Water Development Board (TWDB) (see Section 8.4.2 in Reference 2.4S.3-23). Based on this evaluation, the following three possible PMF scenarios were selected to determine the maximum flood elevation caused by river and stream flooding at the STP 3 & 4 site.

Scenario 1 The flow resulting from the PMF for the drainage area between Mansfield Dam and the Bay City United States Geological Survey (USGS) gauging station (see Figure 2.4S.1-8) combined with an antecedent storm equal to 40% of the PMP occurring over the same drainage area (3555 sq. mi), three days before the PMF. This combined flow is added to the flow release of 90,000 cfs from Mansfield Dam and to the base flow at Bay City to determine the peak PMF flow for this scenario.

The flow release from Mansfield Dam is added to this scenario to accommodate any rainfall contributions upstream of the Mansfield Dam during the PMF event. The “Water Management Plan for the Lower Colorado River Basin” (Reference 2.4S.3-26) states that “if the reservoir level is forecast to exceed 714 ft MSL but not to exceed 722 ft MSL: release will be made at 90,000 cfs” from the Mansfield Dam. It also states that “if the reservoir level is forecast to exceed 722 ft MSL, the Bureau of Reclamation will schedule releases as required for the safety of the structure.”

Scenario 2 The flow resulting from the PMF inflow hydrograph to Mansfield Dam, generated by the PMP storm over the watershed upstream of the dam (from Lake O.H. Ivie to Mansfield Dam), routed through Lake Travis and combined with the flood hydrograph from a sequential storm equal to 40% of the PMP occurring over the drainage area between Mansfield Dam and Bay City (3555 sq. mi), three days after the PMP storm upstream of Mansfield Dam. This combined flow is added to the base flow at Bay City to determine the peak PMF flow for this scenario. The total contributing drainage area for this scenario is about 18,197 sq. mi.

Scenario 3 The flow resulting from the PMF for the entire Lower Colorado River basin area between Lake O.H. Ivie and Bay City (18,197 sq. mi) combined with the flood hydrograph from an antecedent storm equal to the Standard Project Storm (SPS)[1] for the same drainage area occurring three days before the PMF. This combined flow is added to the base flow at Bay City to determine the peak PMF flow for this scenario. This scenario does not account for the storage effect of Lake Travis at Mansfield Dam. The total contributing drainage area for this case is about 18,197 sq. mi.

1 ANSI/ANS 2.8 (Reference 2.4S.3-13) states that the antecedent storm should be equal to 40% of the PMP or the 500-yr storm, whichever is less. However, for this scenario, the SPS event is adopted conservatively as the antecedent storm, considering the fact the SPS event produces a higher flood peak compared to the 500-yr event (see Vol. II-B, Chapter 4, Table VI-7 Reference 2.4S.3-8).

From these three possible PMF flow scenarios, the most critical flow scenario, which produces the highest PMF peak at the Bay City gauging station, is selected to evaluate flooding potential at the STP 3 & 4 site. The Bay City gauging station is located about 18 miles upstream from the STP reservoir makeup pumping facility located on the west bank of Lower Colorado River (see Figure 2.4S.3-1). The discussions on the PMF developments of these three scenarios are given in Section 2.4.S.3.4.2.

In this subsection, failure of the upstream dams was not considered as part of these probable maximum flood scenarios. The implications of potential hydrological dam failures are discussed in Subsection 2.4S.4.

2.4S.3.1 Probable Maximum Precipitation (PMP)

PMP depths for the drainage basins upstream of the STP 3 & 4 site were derived following the procedures described in the National Weather Service (NWS) Hydrometeorological Reports 51 and 52 (HMR 51 and 52) (References 2.4S.3-10 and 2.4S.3-11). The PMP estimates obtained from the HMR 51 and 52 procedures are location-specific and have accounted for orographic and seasonal effects.

The PMP storm spatial distribution, centering, and orientation pattern adopted for the drainage basin upstream of STP 3 & 4 site were determined as follows:

- The storm spatial distribution for the PMP was selected based on the procedures in HMR 51 and 52 (References 2.4S.3-10 and 11), as discussed in detail in Subsection 2.4S.3.4.1.4.
- A critical storm centering approach that produces the largest peak flow rate at the STP 3 & 4 site for the PMP event was determined in a manner that maximizes the volume of precipitation within the basin, as discussed in detail in Subsection 2.4S.3.4.1.4.
- Two different storm orientation patterns were analyzed for the Lower Colorado River basin to derive the most critical PMF flood hydrographs at STP 3 & 4 site. A detailed description of the orientation patterns used for the analyses is provided in Subsection 2.4S.3.4.1.4.

Halff Associates Inc. adopted a storm duration of 96 hours for the rainfall hyetographs used for the Lower Colorado River flood damage evaluation study (see Vol. II-B, Chapter 4, pg. 18 of Reference 2.4S.3-8). The Halff study stated that “a 96-hour storm duration was selected because the upper basin peak could travel to Mansfield Dam during that period and storm events below Mansfield Dam could reach Wharton[2] by the end of the storm event.” The UFSAR also adopted the 96-hour storm duration for the PMP hyetograph used for the STP 1 & 2 site (Reference 2.4S.3-1). Thus, a 96-hour storm duration was selected for the PMP hyetograph developed for the STP 3 & 4 site

2 It should be noted that the difference in drainage areas at Wharton (30,600 sq. mi) and Bay City (30,837 sq. mi) gauging stations is less than 1% of the contributing drainage area at Bay City (Vol. II-B, Chapter 4, Table IV-1 of Reference 2.4S.3-8).

in Subsection 2.4S.3.4.2.1. The time distribution of the 96-hour rainfall hyetographs adopted in the Halff study was used to derive PMP hyetographs for the drainage basins upstream of the STP 3 & 4 site in Subsection 2.4S.3.4.2.1.

Investigations of the occurrence of snow within the Lower Colorado River basin and its effect on flood-producing phenomena indicated that snow melt and antecedent snow pack are not a factor in the production of floods at the STP 3 & 4 site (see Subsection 2.4S.7).

The PMP estimates for the subbasins between Mansfield Dam and the STP 3 & 4 site are provided in Subsection 2.4S.3.4.2.1.

2.4S.3.2 Precipitation Losses

The rainfall-runoff analysis requires estimation of initial rainfall loss and constant rainfall loss rate to determine the direct runoff hydrograph corresponding to the excess rainfall (i.e. total rainfall minus rainfall loss). The initial rainfall loss quantifies the amount of infiltrated or stored rainfall before surface runoff begins. The constant rainfall loss rate determines the rate of infiltration that will occur after the initial loss is satisfied. Conservative assumptions were made for initial and constant loss rates to account for absorption and wet watershed antecedent conditions that would maximize the PMF peak flow, as discussed in detail in Subsection 2.4S.3.4.2.1.

2.4S.3.3 Runoff and Stream Course Models

The PMF hydrograph for the drainage area between Mansfield Dam and Bay City was estimated using the HEC-HMS model developed by Halff Associates Inc. (Reference 2.4S.3-8). This model included the calibrated rainfall losses (i.e. initial loss and constant loss rate) and the Snyder unit hydrograph parameters (i.e. basin lag-time and peaking coefficient) for each of the subbasins located between Mansfield Dam and Matagorda Bay (see Vol. II-B, Chapter 4, Attachment B-1 of Reference 2.4S.3-8). A total of 80 subbasins are included in the lower part of the river basin from Mansfield Dam to Matagorda Bay as shown in Figures 2.4S.3-2(a) and 2(b). The subbasin drainage areas (between Mansfield Dam and Bay City) and the calibrated Snyder unit hydrograph parameters used for the analysis are presented in Subsection 2.4S.3.4.2.1.

Because the Halff HEC-HMS model was calibrated only for floods up to the 100-year storm event, the calibrated unit hydrograph basin lag-time parameter for each subbasin was conservatively decreased by 25% to account for non-linearity effects in the runoff process under extreme flood conditions such as the PMF based on recommendations stated in the United States Army Corps of Engineers EM 1110-2-1417 (Reference 2.4S.3-24). The modified unit hydrograph basin lag-time parameters used for the PMF analysis are presented in Subsection 2.4S.3.4.2.1, for the subbasins from Mansfield Dam to Matagorda Bay.

In the Halff HEC-HMS model, the flow routing from an upstream reach to a downstream reach was performed using the modified Puls method, which defined a storage-outflow rating curve for each of the channel reaches in the model (see Vol. II-

B, Chapter 4, pg. 26 and pg. 29 of Reference 2.4S.3-8). Because the Halff HEC-HMS model (developed for the lower part of river basin) was calibrated only for floods up to the 100-year storm event, it was also necessary to revise the storage-outflow channel rating curves for a few channel reaches between Mansfield Dam and Matagorda Bay to accommodate the PMF conditions. Only three out of a total of 58 channel rating curves needed to be revised. The rating curves for these three channel reaches were extended by linear extrapolation.

2.4S.3.4 Probable Maximum Flood Flow

2.4S.3.4.1 Previous Hydrologic Studies for Lower Colorado River

The following publicly available hydrologic and hydraulic studies performed for the Lower Colorado River basin by Federal, State, and other local agencies were reviewed in detail to determine PMF conditions in Lower Colorado River and their potential to flood the facilities at the STP 3 & 4 site. These studies were listed in the beginning of Subsection 2.4S.3 and are discussed in detail in this subsection.

2.4S.3.4.1.1 PMF Flow Scenarios at the STP 1 & 2 Site – UFSAR

The UFSAR prepared for the existing STP 1 & 2 (Reference 2.4S.3-1) evaluated five hydro-meteorologically critical flow scenarios for the Lower Colorado River and selected among these the most critical PMF flow scenario to determine the maximum flood elevation at the STP 1 & 2 site. This study also included a proposed dam at Columbus Bend that was under consideration in the 1960s. These five PMF flow scenarios are summarized as follows (Reference 2.4S.3-1):

- The Spillway Design Flood (SDF) for the proposed Columbus Bend Dam, which would result from a Probable Maximum Precipitation (PMP) storm on the watershed above the dam, was routed to the STP 1 & 2 site. It was assumed that this event would occur in coincidence with the peak of a Standard Project Flood (SPF) from the 755 sq. mi drainage area between the proposed Columbus Bend Reservoir and the STP 1 & 2 site. It was assumed that the peaks of these two floods would be directly additive and that they would occur simultaneously with a base flow of 50,000 cfs. The peak flow at Bay City for this scenario was estimated to be equal to 958,000 cfs (SDF: 648,000 cfs + SPF: 260,000 cfs + base flow: 50,000 cfs) at the STP 1 & 2 site.

- The PMF for the drainage area between Mansfield Dam and Bay City was assumed to occur three days after the occurrence of the SPF over the same area. A base flow of 50,000 cfs was adopted. The peak flow at Bay City for this scenario was estimated to be equal to 913,000 cfs, which includes a base flow of 50,000 cfs.

- The SDF outflow hydrograph from Mansfield Dam, which results from the PMF inflow hydrograph into Lake Travis caused by a PMP storm on the watershed above the dam, was routed to the STP 1 & 2 site. This was combined with a SPS occurring over the drainage area between Mansfield Dam and the STP 1 & 2 site, three days after the PMP storm producing the Mansfield Dam SDF. It was also assumed that a base flow of 50,000 cfs occurs simultaneously with the resulting flood. The peak flow for this scenario was estimated to be equal to 698,000 cfs, which includes a base flow of 50,000 cfs.

- The PMF for the drainage area between the proposed Columbus Bend Dam and the STP 1 & 2 site was assumed to occur in coincidence with an SPF peak discharge from the proposed dam. It was also assumed that a base flow of 50,000 cfs occurs simultaneously with the resulting flood. The peak PMF for this scenario was estimated to be equal to 894,000 cfs, (PMF: 520,000 cfs + SPF: 324,000 cfs + base flow: 50,000 cfs) at the site.
- A hypothetical PMF for the entire contributing drainage area of the Lower Colorado River basin above the STP 1 & 2 site was assumed, with no credit taken for flood control in the numerous reservoirs upstream from Mansfield Dam, including Lake Travis. The peak PMF for this scenario was estimated to be equal to 1,750,000 cfs.

The PMF flows in the UFSAR for STP 1 & 2 (Reference 2.4S.3-1) were derived based on PMP depths that were calculated according to the procedures outlined in Hydrometeorological Reports 51 and 52 (HMR 51 and 52) (References 2.4S.3-10 and 2.4S.3-11). These reports provide the most up-to-date procedures that replace those originally presented in Hydrometeorological Report 33 (HMR 33) (Reference 2.4S.3-9).

2.4S.3.4.1.2 PMF at Mansfield Dam - USBR and Others

The most recent publicly available PMF inflow hydrograph into Mansfield Dam was established in November 1985 by the USBR (Reference 2.4S.3-2) and was developed using the procedures outlined in HMR 51 and 52 (References 2.4S.3-10 and 2.4S.3-11). According to the USBR study, the peak PMF inflow into Mansfield Dam was estimated to be equal to 931,600 cfs.

In July 1989, ATC Engineering Consultants Inc. (ECI) prepared a dam safety evaluation report (Reference 2.4S.3-5) for Mansfield Dam, using the PMF inflow hydrograph established by the USBR in 1985. This report concluded that when all the bottom outlet gates are closed, the PMF outflow (or SDF) hydrograph has a peak discharge of 602,210 cfs with maximum reservoir water surface elevation at 750.28 ft NGVD29 (also referred to as MSL vertical datum) (Reference 2.4S.3-5).

In March 2003 (Reference 2.4S.3-3), the USBR reviewed the spillway of Mansfield Dam for its design, analysis, and construction features and confirmed the PMF hydrograph with a peak inflow of 931,600 cfs, which was established in 1985 (Reference 2.4S.3-2).

The USBR official website states that the PMF inflow to Mansfield Dam is 931,600 cfs (Reference 2.4S.3-4), i.e. the same as that published by USBR in November 1985.

2.4S.3.4.1.3 Dam Safety Evaluation for Highland Lakes – Freese & Nichols

In August 1992, Freese & Nichols Inc. (Reference 2.4S.3-7) performed a dam safety evaluation study for the six Highland Lakes in the Lower Colorado River, including

Lake Travis at Mansfield Dam, as part of the dam safety compliance study for the Lower Colorado River Authority (LCRA). The watershed area for this study extended from the Lake O.H. Ivie Reservoir to Mansfield Dam and was divided into 41 subbasins (Reference 2.4S.3-7).

The computer models used for this study included HMR 52 for PMP estimates, HEC-1 for rainfall-runoff analysis, and NETWORK for runoff routing (Reference 2.4S.3-7). The subbasin unit hydrographs and rainfall loss rates used in the HEC-1 model and channel roughness coefficients used in the NETWORK model were calibrated based on selected historical flood events in the Lower Colorado River.

In the Freese & Nichols study, PMF levels for the six Highland Lakes, including the PMF at Mansfield Dam, were calculated using the PMP depths derived from procedures outlined in HMR 51 and 52 (References 2.4S.3-10 and 2.4S.3-11). In computing the peak PMF water levels at Mansfield Dam, an antecedent storm event was routed through Lake Travis, before the PMF hydrograph. For this routing, Freese & Nichols (Reference 2.4S.3-7) assumed that the antecedent storm event was equal to 20% of the PMP, which was estimated using the HMR 51 and 52 (References 2.4S.3-10 and 2.4S.3-11).

This study concluded that the PMF outflow hydrograph at Mansfield Dam has a peak discharge of 837,094 cfs and that the maximum water surface elevation at the dam for this flood event was 752.02 ft NGVD29 (Reference 2.4S.3-7).

2.4S.3.4.1.4 Flood Damage Evaluation for Lower Colorado River - Halff Associates

In July 2002, Halff Associates Inc. completed a comprehensive flood damage evaluation study for the Lower Colorado River basin (Reference 2.4S.3-8). The study area extended from Lake O.H. Ivie to the Gulf at Matagorda Bay (see Figure 2.4S.3-3) with a total contributing drainage area of about 18,300 sq. mi. This overall study area was divided into 290 subbasins (see Vol. II-B, Chapter 4, Figures III-2 to III-5 of Reference 2.4S.3-8) to include major reservoirs, major tributary confluences, and the existing USGS stream gauging stations.

The United States Army Corps of Engineer's HEC-HMS model, Version 2.2.2 (Reference 2.4S.3-17) was used for this study as the hydrologic modeling framework to determine frequency flood hydrographs resulting from selected storm events with return periods of 2, 5, 10, 25, 50, 100, and 500-year and SPS. The HEC-HMS models developed for the Halff study were initially calibrated using three historic storm events selected based on availability of adequate rainfall gauge data. The selected three storm events occurred in June 1997, October 1998, and November 2000 (see Vol. II-B, Chapter. 4, pg. 12 of Reference 2.4S.3-8). The calibrated HEC-HMS model parameters included: initial rainfall loss, constant rainfall loss rate, Snyder's basin lag-time, and Snyder's peaking coefficient (see Vol. II-B, Chapter. 4, pg. 16 of Reference 2.4S.3-8). These initially calibrated model parameters were further adjusted to match the peaks of historic flood frequencies estimated at various stream gauging stations located within the study area (see Vol. II-B, Chapter. 4, pg. 23 of Reference 2.4S.3-8).

The synthetic precipitation data used for this study were obtained from Hydro-35 (Reference 2.4S.3-19), TP-40 (Reference 2.4S.3-20), and TP-49 (Reference 2.4S.3-21) for the storm events with return periods of 2, 5, 10, 25, 50, 100, and 500 years. For the SPS event, the SPF Index Rainfall was used. The storm spatial distribution, centering, and orientation pattern adopted for the Halff study were as follows:

- The storm spatial distribution was selected based on the procedures in HMR 51 and 52 (References 2.4S.3-10 and 2.4S.3-11) for storm events with return periods of 2, 5, 10, 25, 50, 100, and 500 years (see Vol. II-B, Chapter. 4, pg. 18 of Reference 2.4S.3-8).
- A critical storm centering approach was used for all the storm events (i.e. 2-, 5-, 10-, 25-, 50-, 100-, and 500-year storm and SPS). Using a trial and error approach, the storm center that produces the largest peak flow rate at a particular point-of-interest (POI) was determined as the critical storm center for that return period (see Vol. II-B, Chapter. 4, pg. 18 of Reference 2.4S.3-8).
- Two different storm orientation patterns were adopted for the Lower Colorado River basin; one for the upper part of the basin and the other for the lower part of the basin, to derive frequency flood hydrographs at different POIs. For example, the storm orientation pattern shown in Figure 2.4S.3-4 was used to estimate flood hydrographs at different POI's in the upper part of the basin, including Mansfield Dam (see Vol. II-B, Chapter. 4, Figure VI-1 of Reference 2.4S.3-8). The orientation pattern shown in Figure 2.4S.3-5 was used to estimate flood hydrographs at different POI's in the lower part of the basin, including Bay City (see Vol. II-B, Chapter. 4, Figure VI-5 of Reference 2.4S.3-8).

Based on the storm orientation pattern adopted for the upper part of the river basin, the peak SPF inflow to Mansfield Dam was estimated to be 801,996 cfs[3], with the critical storm center located at subbasin LR-24 (Vol. II-B, Chapter. 4, Table VI-5 of Reference 2.4S.3-8) for unregulated flow conditions upstream of Mansfield Dam. Based on the same storm orientation pattern, with the critical storm center located at subbasin SS-18, and the unregulated flow conditions, the peak SPF at the Wharton USGS gauging station was estimated to be 423,321 cfs[4], (Vol. II-B, Chapter. 4, Table VI-7 of Reference 2.4S.3-8). The Wharton gauge is located at the Wharton POI shown on Figure 2.4S.3-3. The unregulated flow conditions used to obtain these estimates were based on the assumption that there are no dams or reservoirs in the river basin.

3 The value of 801,996 cfs for the peak of the SPF peak inflow into Mansfield Dam was extracted from the computer files obtained from Halff Associates Inc. In the report documenting this work (Reference 2.4S.3-8) this peak inflow was rounded to 800,000 cfs (see Vol. II-B, Chapter. 4, Table VI-5 of Reference 2.4S.3-8).

4 The value of 423,321 cfs for the peak of the SPF peak at Wharton was extracted from the computer files obtained from Halff Associates Inc. In the report documenting this work (Reference 2.4S.3-8) this peak inflow was rounded to 425,000 cfs (see Vol. II-B, Chapter. 4, Table VI-7 of Reference 2.4S.3-8).

2.4S.3.4.1.5 Review Summary

Table 2.4S.3-1 summarizes the reported PMF and SPF values at Mansfield Dam in the hydrologic studies reviewed in Subsections 2.4S.3.4.1.1 to 2.4S.3.4.1.4.

2.4S.3.4.2 PMF Flow Scenarios at STP 3 & 4

The five flood scenarios of possible PMF flows in the Lower Colorado River that were considered for the STP 1 & 2 (see Subsection 2.4S.3.4.1.1) were first evaluated for their applicability in determining the maximum flood elevation at the STP site for the present conditions. After careful consideration of the hydro-meteorological setting of the region, it was determined that the five flood scenarios considered for the STP 1 & 2 cover the permutation of the possible critical flood events that could occur in the region, thus acceptable for the evaluation of possible extreme flood conditions for the STP 3 & 4.

The first and fourth scenarios considered for STP 1 & 2 (see Subsection 2.4S.3.4.1.1) were eliminated because they include the Columbus Bend Dam that was proposed in the 1960s and which met with opposition by different groups at various times. This dam was also referred to later as the Shaw Bend Dam. Plans for the construction of this dam have been abandoned. This was confirmed by conducting an online search, a search of various sources, as well as inquiries to different engineers of the LCRA, none of which revealed any information regarding continuing plans for the construction of the Columbus Bend Dam. The recently published Region "K" Plan for the Lower Colorado Region in the 2007 State Water Plan also states that "Large local opposition to this project was demonstrated at the various Lower Colorado River Water Planning Group (LCRWPG) public meetings and in correspondence during the 2001 LCRWPG plan preparation." The Planning Group's recommendation in the current water plan is to oppose the potential designation of the Shaw Bend site as a potential reservoir site (see Section 8.4.2 in Reference 2.4S.3-23). Therefore, it was concluded it is not likely that this dam will be constructed in the future.

The three remaining possible PMF flow scenarios in Lower Colorado River that are analyzed for their effects at STP 3 & 4 are as follows:

(1) The PMF for the drainage area between Mansfield Dam and the Bay City USGS gauging station (3555 sq. mi) combined with an antecedent storm equal to 40% of the PMP occurring over the same drainage area, three days before the PMF. This combined flow is added to the flow release from Mansfield Dam and to the base flow at Bay City to determine the peak PMF flow for this scenario (see Subsection 2.4S.3).

(2) The PMF inflow hydrograph to Mansfield Dam, which results from a PMP storm on the watershed upstream of the dam (from Lake O.H. Ivie to Mansfield Dam), routed through Lake Travis and combined with the flood hydrograph from a sequential storm equal to 40% of the PMP occurring over the drainage area between Mansfield Dam and Bay City (3555 sq. mi), three

days after the PMP storm upstream of Mansfield Dam. This combined flow is added to the base flow at Bay City to determine the peak PMF flow for this scenario.

(3) The PMF for the Lower Colorado River basin area between Lake O.H. Ivie and Bay City (18,197 sq. mi) combined with the flood hydrograph from an antecedent storm equal to the SPS over the same area, occurring three days before the PMF. This combined flow is added to the base flow at Bay City to determine the peak PMF flow for this scenario. Conservatively, this scenario does not account for the storage effect of Lake Travis at Mansfield Dam nor any other dam in the Lower Colorado River basin.

From these three possible PMF flow scenarios, the most critical flow scenario, which would produce the highest PMF peak at the Bay City gauging station, is selected to evaluate flooding potential at the STP 3 & 4 site. The Bay City gauging station is located about 18 river miles upstream of the STP Reservoir Makeup Pumping Facility (RMPF) on the west bank of Lower Colorado River (see Figure 2.4S.3-1).

2.4S.3.4.2.1 PMF between Mansfield Dam and Bay City for Scenario 1

For Scenario 1, the peak PMF for the drainage area between Mansfield Dam and Bay City (3555 sq. miles) was calculated by assuming an antecedent storm equal to 40% of the PMP occurs over the same area three days before the PMF event itself and combining those flows with the flow release from Mansfield Dam and the base flow at Bay City (Subsection 2.4S.3). The analysis performed to determine the peak PMF for Scenario 1 is described below.

<u>HEC-HMS Rainfall-Runoff Model</u>

The PMF hydrograph for the drainage area between Mansfield Dam and Bay City (3555 sq. miles) was estimated using the HEC-HMS model developed by Halff Associates Inc. for the lower part of the river basin with the storm orientation pattern shown in Figure 2.4S.3-5 (Subsection 2.4S.3.4.1.4). This model consists of 80 subbasins between Mansfield Dam and Matagorda Bay.

In the Halff HEC-HMS model, the flow routing from an upstream reach to a downstream reach was performed using the modified Puls method, which defines a storage-outflow rating curve for each of the channel reaches in the model. As discussed in Subsection 2.4S.3.3, three storage-outflow rating curves (out of 58) in the original Halff HEC-HMS model were extended to accommodate the PMF conditions. Note that there are nine dams/reservoirs with individual storage capacity in excess of 3000 acre-feet, but none of these reservoirs were included in the Halff HEC-HMS model.

Other input data to the HEC-HMS model included the unit hydrograph, the rainfall hyetograph, and rainfall losses for each of the subbasins in the lower part of the river from Mansfield Dam to Matagorda Bay as described below.

Unit Hydrograph: The HEC-HMS model developed by Halff Associates Inc. included the calibrated Snyder unit hydrograph parameters (i.e. the basin lag-time and peaking coefficient) for each of the subbasins located between Mansfield Dam and Matagorda Bay (Subsection 2.4S.3.4.1.4). As discussed in Subsection 2.4S.3.3, the calibrated Snyder basin lag-time parameter for each of the subbasins was decreased by 25% to account for non-linearity effects in the runoff process under PMF conditions (Reference 2.4S.3-24).

Table 2.4S.3-2 presents the drainage areas and the unit hydrograph parameters for the subbasins from Mansfield Dam to Matagorda Bay extracted from the calibrated HEC-HMS model (see also Vol. II-B, Chapter. 4, Attachment B-1 of Reference 2.4S.3-8) and the Snyder lag times as modified to account for non-linearity effects.

Rainfall Hyetograph: PMP hyetographs were developed for each of the 80 subbasins located within the lower part of the river basin presented in Table 2.4S.3-2, using the same storm spatial distribution and the critical storm centering location adopted by the Halff study (see Subsection 2.4S.3.4.1.4) as follows:

- In the Halff study, the critical storm centering location that produces the largest flow rate at Bay City for the 100-year storm event was found to be at subbasin CC-06, as shown in Figure 2.4S.3-3 (see Vol. II-B, Chapter. 4, Table VI-11 of Reference 2.4S.3-8). Considering the unique elongated shape of the lower part of the Lower Colorado River basin (from Mansfield Dam to Matagorda Bay) and the storm orientation, it is reasonable to assume that the same critical storm centering location can be used for the PMP event.

- The storm spatial distribution pattern adopted by the Halff study was based on the procedures in HMR 52 (Reference 2.4S.3- 11). The same procedures were used to spatially distribute the 96-hour PMP depth at subbasin CC-06 to the remaining subbasins.

- The 96-hour 10-mi^2 PMP depth for subbasin CC-06 was estimated as 55.7 inches by extrapolating data obtained from Figures 18 to 22 in HMR 51 (see Table 2.4S.3-3). The PMP hyetograph for subbasin CC-06 is shown in Figure 2.4S.3-6.

- The PMP hyetographs for the remaining 79 subbasins are the same, except that the rainfall intensity ordinates are multiplied by the ratio of the PMP depth for that subbasin (obtained from Table 2.4S.3-2) to the PMP depth at subbasin CC-06 (55.7 inches, according to Table 2.4S.3-3).

Rainfall Losses: The unit hydrograph approach requires estimation of initial rainfall loss and constant rainfall loss rate to determine the direct runoff hydrograph corresponding to the excess rainfall (i.e. total rainfall minus rainfall loss). The initial rainfall loss quantifies the amount of infiltrated or stored rainfall before surface runoff begins. The constant rainfall rate determines the rate of infiltration that is sustained during the rest of the storm after the initial loss is satisfied.

The PMF peak flow is often insensitive to the initial rainfall loss (Reference 2.4S.3-12); therefore, this value was conservatively set equal to zero for each of the 80 subbasins in the HEC-HMS model (see Table 2.4S.3-2). Reference 2.4S.3-12 also states that "for PMF runoff computations, the soil should be assumed to be saturated with infiltration occurring at the minimum rate applicable to the area-weighted average soil type covering each subbasin." Therefore, based on data provided in Table 8-8.1 of Reference 2.4S.3-12, a minimum uniform rainfall loss rate of 0.05 in/hr was adopted in the model for the PMF analysis (see Table 2.4S.3-2). These conservative values were used in the model to account for absorption and wet watershed antecedent conditions that would maximize the peak PMF discharges for subbasins listed in Table 2.4S.3-2.

<u>Base Flow</u>: The base flow rate at Bay City is estimated in accordance with the procedures in ANSI/ANS-2.8-1992 (Reference 2.4S.3-13), which states that the mean monthly flow should be used as the base flow rate for the PMF analysis. The base flow rate at Bay City was conservatively set equal to the mean monthly average flow of 5200 cfs. This value was selected based on the published USGS mean monthly flow statistics at Austin (08158000), Columbus (08161000), and Bay City (08162500).

<u>Peak PMF Discharge at Bay City for Scenario 1:</u> The PMF hydrograph for the drainage area between Mansfield Dam and Bay City was estimated using the HEC-HMS model obtained from Halff Associates Inc. (Reference 2.4S.3-8) with the input data described above. The peak discharge for this PMF hydrograph (without an antecedent storm event and a base flow) was estimated to be 1,096,807 cfs, (see Figure 2.4S.3-7). As shown in Figure 2.4S.3-7, combining the PMF with an antecedent storm event equal to 40% of the PMP over the same drainage area occurring three days before the PMF event, the flow release of 90,000 cfs from Mansfield Dam (see Subsection 2.4S.3), and the base flow of 5200 cfs gives a peak PMF discharge at Bay City of 1,397,432 cfs (see Figure 2.4S.3-7).

2.4S.3.4.2.2 PMF between Mansfield Dam and Bay City for Scenario 2

For Scenario 2, the PMF inflow hydrograph to Mansfield Dam is routed through Lake Travis and combined with the flood hydrograph from a sequential storm event equal to 40% of the PMP over the drainage area between Mansfield Dam and Bay City and the base flow at Bay City. The sequential storm event occurs three days after the PMP storm that produces the PMF inflow hydrograph into Lake Travis at Mansfield Dam.

The PMF inflow hydrograph into Lake Travis was estimated based on the SPF inflow hydrograph developed for the basin upstream of Mansfield Dam with unregulated flow conditions as reported in the Halff study (see Subsection 2.4S.3.4.1.4). The PMF inflow was taken as equal to two times the SPF inflow into Lake Travis at Mansfield Dam. This assumption is based on guidelines given in the United States Army Corps of Engineers E.M. 1110-2-1411 (Reference 2.4S.3-14), which states that the SPF is usually equal 40 to 60% of the PMF for the same basin. A ratio of 50% is adopted in this PMF analysis.

The critical storm centering location for this SPF inflow hydrograph at Mansfield was found to be at subbasin LR-24, as shown in Figure 2.4S.3-3 (Vol. II-B, Chapter. 4, Table VI-5, Reference 2.4S.3-8).

Routing of PMF Hydrograph through Lake Travis: The SPF inflow hydrograph at Mansfield Dam was routed through Lake Travis, using the United States Army Corps of Engineers HEC-1 model (Reference 2.4S.3-15), in order to establish the antecedent water level conditions in the reservoir. The SPF inflow hydrograph was assumed to occur three days prior to the routing of the PMF inflow hydrograph that was estimated as equal to two times the SPF inflow. The input data used in the HEC-1 model for the reservoir routing analysis are briefly described below:

- The initial reservoir water level prior to the routing of the SPF inflow hydrograph was set at elevation 681 ft NGVD29, i.e. the elevation of the reservoir conservation pool (see Table 2.4S.3-4).
- The reservoir elevation-storage data up to El. 740 ft NGVD29 were obtained from the Halff Reservoir Operation Model HEC-5 (see Vol. II-B, Chapter 5, Reference 2.4S.3-8) and are presented in Table 2.4S.3-5. The storage values above El. 740 ft NGVD29 were estimated by logarithmic extrapolation of the Halff data.
- The pertinent dam and spillway outlet data were obtained from various USBR publications (References 2.4S.3-4, 2.4S.3-5, and 2.4S.3-16), the Freese & Nichols study (Reference 2.4S.3-7), and from the Halff study (Reference 2.4S.3-8) and are presented in Table 2.4S.3-4. The main spillway at Mansfield Dam is an uncontrolled ogee crest spillway with a 700 ft clear length and crest at El. 714 ft NGVD29 (Reference 2.4S.3-5). The low level outlets consist of twenty-four 102-in diameter conduits through the concrete section of the dam controlled by gates. The centerline elevation of the inlets to the conduits is at El. 540 ft NGVD29 (Reference 2.4S.3-5).
- The main spillway rating curve for Mansfield Dam was computed from spillway capacity data given in References 2.4S.3-4, 2.4S.3-8, and 2.4S.3-16. The discharge coefficient for the spillway is $C = 4.0$ in the expression $Q = CLH^{1.5}$, where Q is the spillway discharge, L is the spillway length, and H is the head over the spillway crest. This value is based on the model test result of $C = 3.93$ at the spillway design head and calculation of C value at other heads in accordance with data in Reference 2.4S.3-22.
- A rating curve for the 24 low level outlets was also developed from data given in References 2.4S.3-4, 2.4S.3-8, and 2.4S.3-16. The low level outlets were treated as orifices in the HEC-1 model with a discharge coefficient of 0.87. The total discharge from all 24 outlet conduits with Lake Travis at El. 714 ft NGVD29 is 126, 000 cfs (Reference 2.4S.3-5).
- The combined inflow hydrograph (SPF + PMF) into the Lake Travis reservoir was estimated by adding the SPF hydrograph ordinates to the PMF hydrograph ordinates, after shifting the latter by three days, as presented in Figure 2.4S.3-8. This figure also includes the routed outflow hydrograph for the combined SPF and PMF event.

As shown in Figure 2.4S.3-8, the peak PMF inflow into Lake Travis at Mansfield Dam was estimated as 1,603,992 cfs (i.e. twice the peak SPF inflow of 801,996 cfs)

(Reference 2.4S.3-8). The reservoir routing analysis showed that the peak PMF outflow at Mansfield Dam is about 944,138 cfs (see Figure 2.4S.3-8).

The HEC-1 routing with this very conservative estimate of the PMF inflow shows that Mansfield Dam would be overtopped. For the purpose of the PMF analysis, it was assumed that Mansfield Dam would not fail. The dam break analysis for Mansfield Dam is addressed in Subsection 2.4S.4. The peak outflow from Mansfield Dam obtained with the HEC-1 routing (944,138 cfs) exceeds all published values reviewed (see Table 2.4S.3-1) for the STP 3 & 4 and provides a conservative value for the peak outflow from Mansfield Dam.

Consequently, the PMF inflow and outflow hydrographs at Mansfield Dam developed as described above should only be used for the intended purpose of STP 3 & 4. These results are meant to provide conservative, i.e. high, estimates of maximum water levels in the vicinity of STP 3 & 4 site. These results should not be used for any other purpose; neither should any conclusions be drawn from these results regarding the potential flooding of areas downstream of Mansfield Dam.

Peak PMF Discharge at Bay City for Scenario 2: The peak PMF discharge at Bay City was calculated by adding the peak PMF outflow at Mansfield Dam (944,138 cfs) (see Figure 2.4S.3-8) to the peak of the 40% PMP hydrograph for the drainage area between Mansfield Dam and Bay City (303,277 cfs) (see Figure 2.4S.3-7) plus the base flow of 5200 cfs (see Subsection 2.4S.3.4.2.1). This approach provides a very conservative estimate for the peak PMF discharge at Bay City (1,252,615 cfs), because it does not account for the attenuation of the peak outflow from Mansfield Dam as the flood wave travels down the 290 mile long reach in the Lower Colorado River between Mansfield Dam and Bay City.

2.4S.3.4.2.3 PMF between Lake O.H. Ivie and Bay City for Scenario 3

For Scenario 3, the peak PMF for the Lower Colorado River basin area from Lake O.H. Ivie and Bay City was estimated by assuming that the SPF occurs over the same basin area three days before the PMF event and combining those flows with the base flow at Bay City. This scenario does not account for the storage effect of Lake Travis at Mansfield Dam or any other dam in the Lower Colorado River basin.

The PMF hydrograph for this scenario was estimated based on the SPF hydrograph that was developed at Wharton by the Halff study (Reference 2.4S.3-8) for unregulated flow conditions in Lower Colorado River Basin. The critical storm centering location for this SPF hydrograph at Wharton is found to be at subbasin SS-18 (see Vol. II-B, Chapter. 4, Table VI-7 of Reference 2.4S.3-8), which is located in the upper portion of the Lower Colorado River basin, as shown in Figure 2.4S.3-3. The SPF peak discharge at Wharton was estimated as 423,321[5] cfs (Subsection 2.4S.3.4.1.4).

5 The value of 423,321 cfs for the peak of the SPF peak at Wharton was extracted from the computer files obtained from Halff Associates Inc. In the report documenting this work (Reference 2.4S.3-8) this peak inflow was rounded to 425,000 cfs (Reference 2.4S.3-8, Vol. II-B, Chapter. 4, Table VI-7 of).

Estimation of PMF Hydrograph at Bay City: The SPF hydrograph at Wharton (with a peak discharge of 423,321 cfs) was used to estimate the SPF hydrograph at Bay City. The ordinates of SPF hydrograph at Wharton were multiplied by the ratio of the drainage area at Bay City (30,837 sq. mi) over the drainage area at Wharton (30,600 sq. mi) to estimate the SPF hydrograph at Bay City. The SPF peak discharge at Bay City was estimated to be about 426,000 cfs. The drainage areas at Wharton and Bay City were obtained from the Halff study (Vol. II-B, Chapter 4, Table IV-1 of Reference 2.4S.3-8). The required PMF hydrograph at Bay City for Scenario 3 was estimated by assuming that the PMF is equal to twice the SPF at Bay City. This assumption is based on guidelines in Engineering Manual EM 1110-2-1411 (Reference 2.4S.3-14). The PMF peak discharge at Bay City was estimated to be 853,200 cfs (see Figure 2.4S.3-9).

Peak PMF Discharge at Bay City for Scenario 3: The PMF hydrograph developed for the Lower Colorado River basin from Lake O.H. Ivie to Bay City has a peak discharge of 853,200 cfs at Bay City (see Figure 2.4S.3-9) without the SPF event and base flow. As shown in Figure 2.4S.3-9, adding the SPF event with a peak discharge of 426,600 cfs over the same area, occurring three days before the PMF event and the base flow of 5,200 cfs at Bay (see Subsection 2.4S.3.4.2.1), produces a peak PMF at Bay City equal to 994,060 cfs (see Figure 2.4S.3-9).

2.4S.3.4.3 Most Critical PMF Scenario at Bay City

The analyses discussed in Subsections 2.4S.3.4.2.1, 2.4S.3.4.2.2, and 2.4S.3.4.2.3 show that Scenario 1 produces the highest peak PMF at Bay City (see Table 2.4S.4.3-6). The highest flood peak at Bay City is caused by the PMP for the drainage area between Mansfield Dam and the Bay City combined with an antecedent storm equal to 40% of the PMP occurring over the same drainage area, the flow release of 90,000 cfs from Mansfield Dam, and the base flow of 5200 cfs. Therefore, the peak flow of 1,397,432 cfs for Scenario 1 is used as the most critical PMF scenario to evaluate potential flooding at the STP 3 & 4 site.

2.4S.3.5 Water Level Determinations

The maximum still water surface elevation at the STP 3 & 4 site for the peak PMF discharge of 1,397,432 cfs was calculated using the United States Army Corps of Engineer's HEC-RAS hydraulic model, Version 3.1.3 (Reference 2.4S.3-18). The HEC-RAS model for the STP 3 & 4 site was developed on the basis of topographic data and hydraulic characteristics such as Manning's roughness coefficients that were established for the Halff's flood damage evaluation study (Reference 2.4S.3- 8).

2.4S.3.5.1 Halff HEC-RAS Hydraulic Model - Bay City to Matagorda Bay

The Halff HEC-RAS model (from Bay City to Matagorda Bay), that was developed for the Halff's flood damage evaluation study used the most recent channel and floodplain topographic information obtained from LCRA and the United States Army Corps of Engineers (USACOE). The required channel topographic data were field-surveyed and provided by USACOE. The floodplain topographic data obtained from LCRA included aerial digital ortho-photographs, digital contour maps (2 foot intervals), and USGS 30-m National Elevation Dataset (NED) Digital Elevation Model (DEM) data.

The 30-m DEM data were used only to fill a 0.5 mile buffer zone area outside the 500-year floodplain that was mapped using the aerial digital data (Vol.1, pg. 20 of Reference 2.4S.3-8).

The Halff HEC-RAS model from Bay City to Matagorda Bay covers approximately a reach length of 24 miles and includes two bridge crossings, one at the Missouri Pacific Railroad (RS 1350+15.3) and another at the FM 521 roadway (RS 843+40.0). The upstream-most cross-section in the Halff model is located at the Bay City USGS gauging station (RS 1665+21.6). The downstream-most cross-section (RS 383+64.5) in the model is located about 4600 ft upstream of the intersection of Lower Colorado River and the Intra-Coastal Waterway (RS 337+90) (see Vol. II-C, Chapter 6, Table I-1 of Reference 2.4S.3). Table2.4S.4.3-7 lists the key cross-sections in the Halff HEC-RAS model, which include two bridge crossings and 68 channel cross-sections.

The initial Manning's roughness coefficients used in the Halff HEC-RAS model were estimated from the USGS National Land Cover Dataset coverage and then adjusted using aerial photographs (see Vol. II-C, Chapter 6, Table III-2 of Reference 2.4S.3-8). During the model calibration by Halff Associates, the roughness coefficients were adjusted in the model to match historical flood levels. The calibrated Manning's roughness coefficients used in model are 0.035 for the river channel, 0.045-0.05 for the overbank, and 0.085-0.095 for the floodplain.

2.4S.3.5.2 Extension of Cross-sections for the PMF Event

A review of the geometry data used in the Halff HEC-RAS model showed that the cross-sections need to be extended to more accurately reflect the potential increase in the width of the floodplain during the passage of a PMF event. The cross-section data used in Halff's HEC-RAS model was therefore extended, as shown in Figure 2.4S.3-10, to cover a larger floodplain area between the most downstream cross-section (RS 383+64.5) and the STP 3 & 4 site (RS 964+99.7). Because the stretch of the Colorado River from the site to Matagorda Bay is in a sub-critical flow regime, it is not necessary to extend the cross-sections any further upstream from the STP 3 & 4 site because the flood elevation at the site depends only on conditions downstream from the STP 3 & 4 site.

A total of 32 cross-sections were extended (between RS 383+64.5 and RS 964+99.7) for a distance up to about 19 miles towards the east of the Lower Colorado River to near Caney Creek. The source maps used for the extension of these cross sections were high-resolution digital raster graphic (DRG) scans of the USGS 7.5-minute quadrangles[6]. To be conservative, these cross-sections were not extended to the west of the Colorado River.

2.4S.3.5.3 HEC-RAS Hydraulic Model for STP 3 & 4

The HEC-RAS hydraulic model (Version 3.1.3) for the STP 3 & 4 site was developed using the above extended cross-sections (from RS 383+64.5 to RS 964+99.7) and

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6 The vertical datum for the USGS 7.5-minute quadrangles is referenced to NGVD29. This datum is adjusted to match NAVD88 that was used as the vertical datum for the Halff's cross-sections.

Manning's roughness coefficients adjusted for PMF flow conditions. As the flow depth increases, the flow encounters larger size obstructions, e.g. shrubs, trees, etc, which effectively increase the roughness of the floodplain. For this purpose the calibrated Manning's roughness coefficients used in the Halff HEC-RAS model (see Subsection 2.4S.3.5.1) were increased by 20% for the postulated PMF flow condition to provide a conservative estimate of the maximum stream flooding elevation at the site.

The HEC-RAS model developed for the STP 3 & 4 covers an approximate reach length of 11 miles and includes a bridge crossing at the FM 521 (RS 843+40.0). Incorporation of this bridge crossing in the model gives a conservative (i.e. higher) estimate for the maximum flood level. The upstream-most cross-section in this model is located at RS 964+99.7 and the downstream-most cross-section is at RS 383+64.5.

2.4S.3.5.3.1 Model Boundary Conditions

Under PMF flow conditions, the water level in the river at the downstream-most cross-section (RS 383+64.5) is not influenced by tidal effects. Therefore, normal depth for an estimated channel slope of 0.0001 is the appropriate boundary condition to use at the downstream-most cross-section of the model that is located approximately 7.3 mile upstream from the shoreline of the Gulf of Mexico (see Table 2.4S.4.3-7).

Using the HEC-RAS model developed for the STP 3 & 4 site, the normal depth at the downstream boundary (RS 383+64.5) was estimated to be equal to 17.5 ft NAVD88 for the peak PMF discharge of 1,397,432 cfs (see Figure 2.4S.3-11) with a steady state model simulation. This calculation was made using Manning's *n* values equal to 1.2 times those used in the Halff HEC-RAS model (see Subsection 2.4S.3.5.1) to provide a conservative upper bound flood level at the site as a result of a PMF event.

Using the same Manning's *n* values as those used in the Halff HEC-RAS model (see Subsection 2.4S.3.5.1), the normal depth at the downstream boundary (RS 383+64.5) was estimated to be equal to 16.2 ft NAVD88 for the peak PMF discharge of 1,397,432 cfs (see Figure 2.4S.3-12).

2.4S.3.5.3.2 PMF Still Water Surface Elevation at STP 3 & 4

As shown in Figure 2.4S.3-11, the maximum PMF still water surface elevation at the STP 3 & 4 site (RS 891+46.0) for the normal depth boundary condition was estimated to be equal to 26.1 ft NAVD88 (26.3 ft NGVD29), which is lower than the design plant grade elevation of 35 ft NGVD29 for safety related structures. The PMF water level of 26.1 ft NAVD88 (26.3 ft NGVD29) at STP 3 & 4 was obtained using conservative Manning's *n* values equal to 1.2 times those used in the original Halff model.

The PMF still water surface profile obtained using the same Manning's *n* values as those used in the Halff model is shown in Figure 2.4S.3-12. In this case, the maximum PMF still water surface elevation at the STP 3 & 4 site (RS 891+46.0) was estimated as 24.8 ft NAVD88 (25.0 ft NGVD29).

PMF water levels at two selected cross-sections: the downstream boundary (RS 383+64.5) and the STP 3 & 4 site (RS 891+46.0) are shown in Figure 2.4S.3-13 (with

Manning's *n* values equal to 1.2 times in the Halff model) and Figure 2.4S.3-14 (with same Manning's *n* values used in the Halff model).

2.4S.3.6 Coincident Wind Wave Activity

The flooding resulting from dam failures upstream of the STP 3 & 4 site was found to be more critical than that resulting from the PMF. For example, the calculated maximum still water level at the STP site due to a domino-type failure of the upstream dams would be at 28.4 ft NAVD88 (28.6 ft NGVD29), (Subsection 2.4S.4.2.15), which is about 2.3 ft higher than the calculated maximum still water level of 26.1 ft NAVD88 (26.3 ft NGVD29) resulting from the PMF. Coincident wind wave activity was therefore considered for flooding resulting from dam failures only (Subsection 2.4S.4.3.1).

2.4S.3.7 References

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2.4S.3-15 “HEC-1 Flood Hydrograph Package, User’s Manual, Version 4.0,” United States Army Corps of Engineers, September 1990.

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2.4S.3-21 “Technical Paper No. 49, Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States,” United States Department of Commerce, Weather Bureau, May 1964.

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2.4S.3-23 “2006 Region K Water Plan for the Lower Colorado Regional Water Planning Group,” Texas Water Development Board (TWDB), January 2006.

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2.4S.3-25 “Engineering Data on Dams and Reservoirs in Texas,” Report 126, Part III, Texas Water Development Board (TWDB), February 1971.

2.4S.3-26 “Water Management Plan for the Lower Colorado River Basin,” Lower Colorado River Authority (LCRA), March 1999.

Table 2.4S.3-1 PMF and SPF values at Mansfield Dam

Hydrologic Study Reviewed	PMF		SPF	
	Inflow (cfs)	Outflow (cfs)	Inflow (cfs)	Outflow (cfs)
UFSAR for STP 1 & 2 (Reference 2.4S.3-1)	957,000 [1]	706,000 [1]	-	-
USBR and Others (References 2.4S.3-2, 2.4S.3-3, 4, 2.4S.3-5, and 2.4S.3-6)	931,600 [1]	602,210 [1]	-	-
Freese Nichols Inc. (Reference 2.4S.3-7)	-	837,094 [1]	-	-
Halff Associates Inc. (Reference 2.4S.3-8)	-	-	801,996 [2],[3]	-

[1] Estimated based on HMR 52 (Reference 2.4S.3-11).

[2] Estimated based on Engineering Manual 1110-2-1411 (Reference 2.4S.3-14).

[3] The value of 801,996 cfs for the peak of the SPF peak inflow into Mansfield Dam was extracted from the computer files obtained from Halff Associates Inc. In the report documenting this work (Reference 2.4S.3-8) this peak inflow was rounded to 800,000 cfs (see Vol. II-B, Chapter. 4, Table VI-5 of Reference 2.4S.3-8).

Table 2.4S.3-2 Drainage Areas, Unit Hydrograph Parameters, Rainfall Loss Rates, and PMP Depths for Subbasins from Mansfield Dam to Matagorda Bay

Sub-basin Name	Drainage Area [1] (sq. mi.)	Percent Impervious [1]	Snyder Peaking Coefficient [1]	Snyder Time Lag [2] (hours)	Initial Rainfall Loss [3] (in)	Constant Rainfall Loss [3] (in/hr)	PMP Depth [4] (in)
AL-16	25.2	21.12	0.7	1.22	0	0.05	39.8
AL-17	43.7	5.39	0.5	2.17	0	0.05	41.2
AL-18	1.4	9.56	0.5	0.71	0	0.05	43.1
AL-19	22.7	2.88	0.5	1.73	0	0.05	41.2
AL-20	9.2	2.08	0.5	1.04	0	0.05	42.4
AL-21	14.1	7.95	0.5	1.01	0	0.05	43.8
AL-22	8.7	10.69	0.5	1.04	0	0.05	44.1
AL-23	89.6	0.44	0.8	5.07	0	0.05	37.9
AL-24	17.8	0.00	0.8	2.59	0	0.05	41.9
AL-25	9.4	7.39	0.8	1.97	0	0.05	42.3
AL-26	3.1	8.46	0.5	0.98	0	0.05	44.3
AL-27	28.6	23.06	0.5	1.61	0	0.05	44.7
AL-28	1.9	27.99	0.5	0.80	0	0.05	46.5
AL-29	20.6	19.68	0.6	3.89	0	0.05	46.9
AL-30	51.6	12.44	0.7	3.19	0	0.05	43.3
AL-31	4.8	3.09	0.6	3.56	0	0.05	47.4
AL-32	14.0	16.05	0.6	3.86	0	0.05	47.6
AL-33	6.6	12.85	0.6	4.15	0	0.05	49.9
AL-34	104.6	0.33	0.7	3.62	0	0.05	34.7
AL-35	19.0	0.00	0.7	2.26	0	0.05	36.4
AL-36	43.7	0.94	0.8	3.70	0	0.05	36.2
AL-37	66.7	0.23	0.8	4.00	0	0.05	36.7
AL-38	89.7	3.91	0.8	4.64	0	0.05	41.7
AL-39	21.4	7.59	0.6	4.67	0	0.05	47.6
CC-01	6.3	6.12	0.6	3.71	0	0.05	51.0
CC-02	41.5	0.62	0.6	3.52	0	0.05	42.4
CC-03	33.8	6.51	0.6	4.50	0	0.05	48.6
CC-04	25.6	4.07	0.6	4.66	0	0.05	53.1
CC-05	55.0	0.86	0.6	5.53	0	0.05	48.8
CC-06	22.6	3.86	0.6	4.99	0	0.05	55.7
CC-07	163.7	0.61	0.6	7.76	0	0.05	44.5
CC-08	17.4	0.78	0.6	4.22	0	0.05	51.4

Table 2.4S.3-2 Drainage Areas, Unit Hydrograph Parameters, Rainfall Loss Rates, and PMP Depths for Subbasins from Mansfield Dam to Matagorda Bay (Continued)

Sub-basin Name	Drainage Area [1] (sq. mi.)	Percent Impervious [1]	Snyder Peaking Coefficient [1]	Snyder Time Lag [2] (hours)	Initial Rainfall Loss [3] (in)	Constant Rainfall Loss [3] (in/hr)	PMP Depth [4] (in)
CC-09	3.0	9.05	0.6	3.56	0	0.05	55.2
CC-10	62.6	0.69	0.6	4.82	0	0.05	43.3
CC-11	47.0	0.63	0.6	5.15	0	0.05	48.1
CC-12	52.2	4.79	0.6	4.76	0	0.05	50.2
CC-13	28.7	4.16	0.45	5.42	0	0.05	51.9
CC-14	39.6	0.58	0.45	6.66	0	0.05	46.7
CC-15	56.0	0.68	0.45	5.75	0	0.05	44.3
CC-16	34.8	0.80	0.45	5.22	0	0.05	50.2
CC-17	12.1	0.31	0.45	5.02	0	0.05	50.5
CC-18	137.5	0.52	0.45	7.08	0	0.05	43.5
CC-19	65.4	0.62	0.45	5.83	0	0.05	45.0
CC-20	5.5	1.03	0.45	4.66	0	0.05	50.5
CC-21	102.4	1.55	0.45	6.20	0	0.05	50.5
CC-22	41.8	3.91	0.3	4.65	0	0.05	48.1
CC-23	42.3	1.52	0.3	5.12	0	0.05	45.9
CC-24	17.4	2.94	0.3	4.43	0	0.05	46.0
CC-25	118.1	1.25	0.3	5.90	0	0.05	47.4
CC-26	38.7	3.65	0.3	4.78	0	0.05	45.7
CC-27	125.1	1.34	0.3	6.24	0	0.05	43.6
CC-28	28.1	2.85	0.3	4.35	0	0.05	44.3
CC-29	3.0	9.08	0.3	1.16	0	0.05	43.9
CC-30	91.6	1.04	0.9	11.10	0	0.05	41.4
CC-31	94.2	1.15	0.3	7.06	0	0.05	43.1
CC-32	103.2	5.63	0.3	5.51	0	0.05	43.1
CC-33	82.1	2.49	0.3	6.08	0	0.05	41.4
CC-34	78.6	2.2	0.3	6.50	0	0.05	39.0
CC-35	80.7	1.63	0.4	3.98	0	0.05	40.9
CC-36	95.9	1.11	0.4	4.33	0	0.05	40.7
CC-37	75.3	0.68	0.4	4.37	0	0.05	40.2
CC-38	63.4	1.12	0.3	5.93	0	0.05	38.8
LC-01	94.5	6.99	0.3	6.44	0	0.05	37.1
LC-02	110.8	3.75	0.3	6.57	0	0.05	36.7

Table 2.4S.3-2 Drainage Areas, Unit Hydrograph Parameters, Rainfall Loss Rates, and PMP Depths for Subbasins from Mansfield Dam to Matagorda Bay (Continued)

Sub-basin Name	Drainage Area [1] (sq. mi.)	Percent Impervious [1]	Snyder Peaking Coefficient [1]	Snyder Time Lag [2] (hours)	Initial Rainfall Loss [3] (in)	Constant Rainfall Loss [3] (in/hr)	PMP Depth [4] (in)
LC-03	62.8	13.38	0.3	8.95	0	0.05	35.5
LC-04	63.2	8.35	0.3	6.50	0	0.05	34.2
LC-05	32.1	7.73	0.3	7.54	0	0.05	33.5
LC-06	29.7	10.44	0.3	5.72	0	0.05	32.8
LC-07	35.6	5.43	0.3	5.52	0	0.05	32.3
LC-08	29.3	4.40	0.3	5.63	0	0.05	32.9
LC-09	33.4	3.38	0.3	4.68	0	0.05	32.8
LC-10	20.4	11.78	0.3	5.27	0	0.05	31.9
LC-11	21.6	7.36	0.3	3.35	0	0.05	31.4
LC-12	50.3	4.59	0.3	4.28	0	0.05	31.9
LC-13	30.2	10.21	0.3	3.42	0	0.05	31.2
LC-14	31.0	7.63	0.5	3.50	0	0.05	30.9
LC-15	27.3	2.72	0.7	2.33	0	0.05	30.7
LC-16	38.4	7.07	0.7	1.94	0	0.05	30.4
LC-17	34.8	33.89	0.7	2.35	0	0.05	30.2
LC-18	2.6	60.91	0.7	1.09	0	0.05	30.0

[1] Drainage areas, percentage impervious values, and calibrated Snyder peaking coefficients are extracted from the Halff HEC-HMS model (Vol. II-B, Chapter. 4, Attachment B-1 of Reference 2.4S.3-8).

[2] Snyder lag time values given here account for the non-linearity effect in the runoff process during a PMF event. The calibrated Snyder lag time parameters extracted from the Halff HEC-HMS model are decreased by 25% to obtain these values.

[3] Initial rainfall loss and constant rainfall loss rate values are obtained from Reference 2.4S.3-12 for the PMF conditions.

[4] Estimated PMP depths used for the PMF calculations at STP 3 & 4 site (see Subsection 2.4S.3.4.2.1).

Table 2.4S.3-3 10 sq. miles PMP Depth at Subbasin CC-06

Duration (hour)	PMP Depth (inches)	Remarks
6	31.0	Figure 18, HMR 51
12	37.5	Figure 19, HMR 51
24	44.8	Figure 20, HMR 51
48	50.0	Figure 21, HMR 51
72	53.1	Figure 22, HMR 51
96	55.7	Extrapolated

Table 2.4S.3-4 Dam and Spillway Outlet Data for Lake Travis Reservoir

Description	Elevation (ft)	Length/or Diameter [3] (ft)	Reservoir Storage (acre-ft)
Low level outlet	540	24 102-in conduits	32,500 [1]
Conservation pool	681	n/a	1,132,172 [1]
Uncontrolled ogee spillway crest	714	700 clear opening	1,879,794 [1], [4]
Dam crest (concrete section)	750	2710	3,125,683 [2], [4]
Floodwall crest	754.1	4393 [5]	3,308,030 [2]

[1] Elevation vs. Storage data (from El. 540 ft to El. 714 ft NGVD29) are obtained from the Halff Reservoir Operation Model HEC-5 (see Vol. II-B, Chapter 5, Reference 2.4S.3-8).

[2] Storage values are estimated by logarithmic extrapolation of elevation-storage data from El. 691 ft to El. 740 ft NGVD29 (see Table 2.4S.3-5).

[3] Elevation, length, and diameter values are obtained from Halff (Reference 2.4S.3- 8), USBR (Reference 2.4S.3-16), and TWDB (Reference 2.4S.3- 25).

[4] Reference 2.4S.3-5 states that at El. 714 ft NGVD29, the reservoir storage capacity is equal to 1,953,000 acre-ft and at El. 750 ft NGVD29, the storage capacity is equal to 2,893,800 acre-ft.

[5] Floodwall length is set equal to the length of the concrete dam (i.e. 5093 ft – 700 ft), where 5093 ft is the total length of the dam section as per USBR (Reference 2.4S.3-16).

Table 2.4S.3-5 Elevation-Storage Data for Lake Travis Reservoir at Mansfield Dam

Reservoir Water Surface Elevation (in feet, NGVD29 Datum [3])	Reservoir Storage Volume (acre-ft)
536	18,270 [1]
630	436,502 [1]
650	652,977 [1]
670	939,110 [1]
691	1,329,593 [1]
710	1,772,913 [1]
722	2,109,176 [1]
732	2,428,210 [1]
740	2,710,598 [1]
750	3,125,683 [2]
760	3,587,326 [2]

[1] Elevation vs. Storage data (from El. 536 ft to El. 740 ft NGVD29) are obtained from the Halff Reservoir Operation Model HEC-5 (see Vol. II-B, Chapter 5, Reference 2.4S.3-8).

[2] Storage values are estimated by logarithmic extrapolation of elevation-storage data from El. 691 ft to El. 740 ft NGVD29.

[3] At Lake Travis reservoir, NAVD88 ft = NGVD29 ft + 0.22 ft.

Table 2.4S.3-6 Estimated Peak PMF at Bay City for STP 3 & 4

Description of the PMF Flow Scenario	Peak PMF at Bay City (cfs)
Scenario 1: PMF for the drainage area between Mansfield Dam and the Bay City, combined with the flood hydrograph from an antecedent storm equal to 40% of the PMP occurring over the same drainage area, three days before the PMF, a flow release of 90,000 cfs from Mansfield Dam, and a base flow of 5200 cfs.	1,397,432
Scenario 2: PMF inflow hydrograph to Mansfield Dam routed through Lake Travis and combined with the flood hydrograph from a sequential storm equal to 40% of the PMP occurring over the drainage area (Mansfield Dam to Bay City), three days after the PMP storm upstream of Mansfield Dam and a base flow of 5200 cfs	1,252,615
Scenario 3: PMF for the entire Lower Colorado River basin area between Lake O.H. Ivie and Bay City combined with the flood hydrograph from an antecedent storm equal to the SPS over the same area, occurring three days before the PMF and a base flow of 5200 cfs.	994,060

Table 2.4S.3-7 Location Description for Key Cross Sections in the HEC-RAS Model

Location Description of Cross-section	HEC-RAS Cross-section No.	River Station (feet)	River Station (miles)
Bay City USGS Station	1	RS 1665+21.6	31.54
Bridge Missouri Pacific Railroad	16	RS 1350+15.3	25.57
STP 3 & 4 Site	43	RS 891+46.0	16.89
Bridge at FM 521	47	RS 843+40.0	15.97
4600 ft upstream from Intra-Coastal Waterway	70	RS 383+64.5	7.27

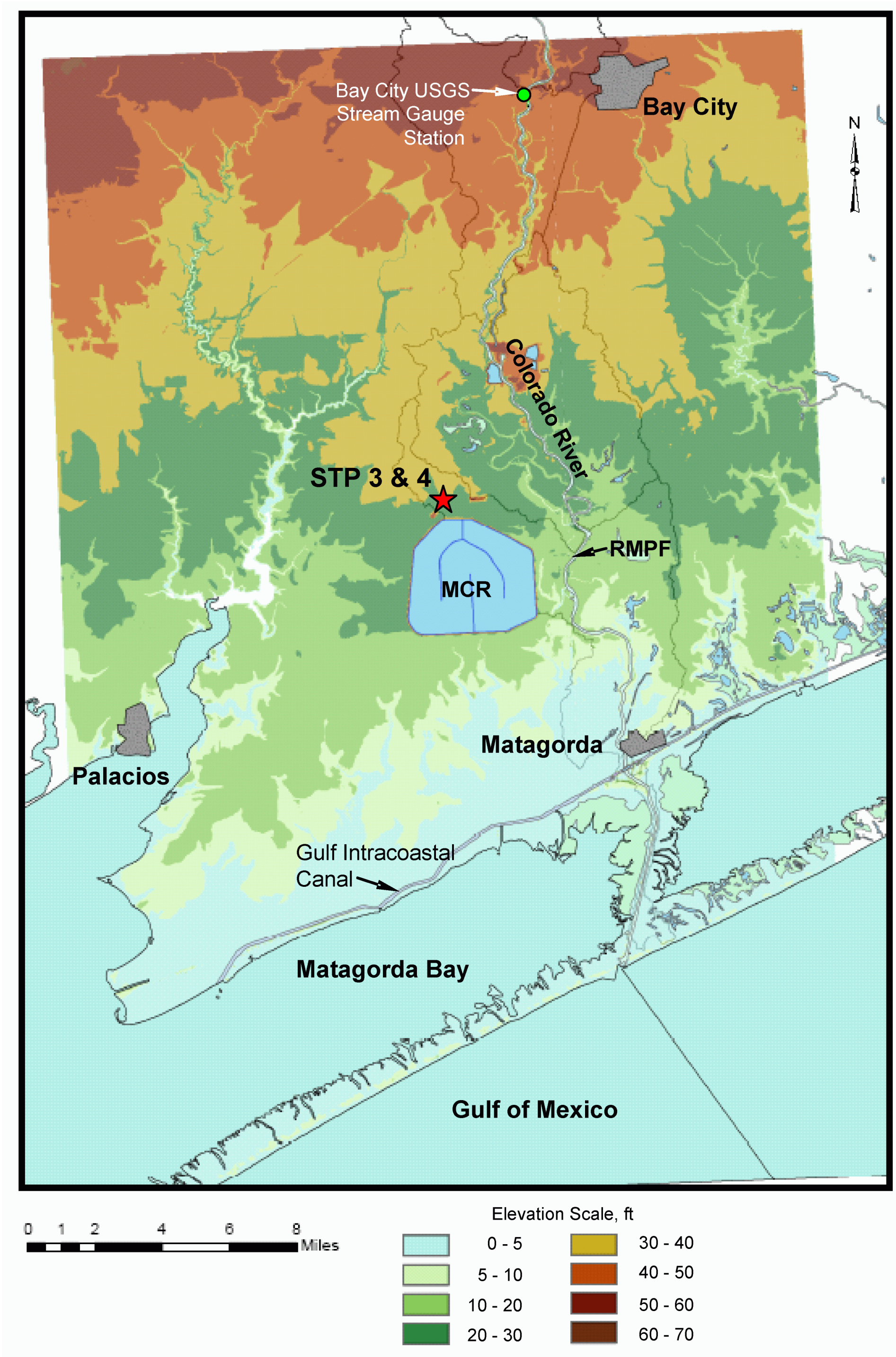

Figure 2.4S.3-1 General Location of STP 3 & 4 Site in the Lower Colorado River Basin

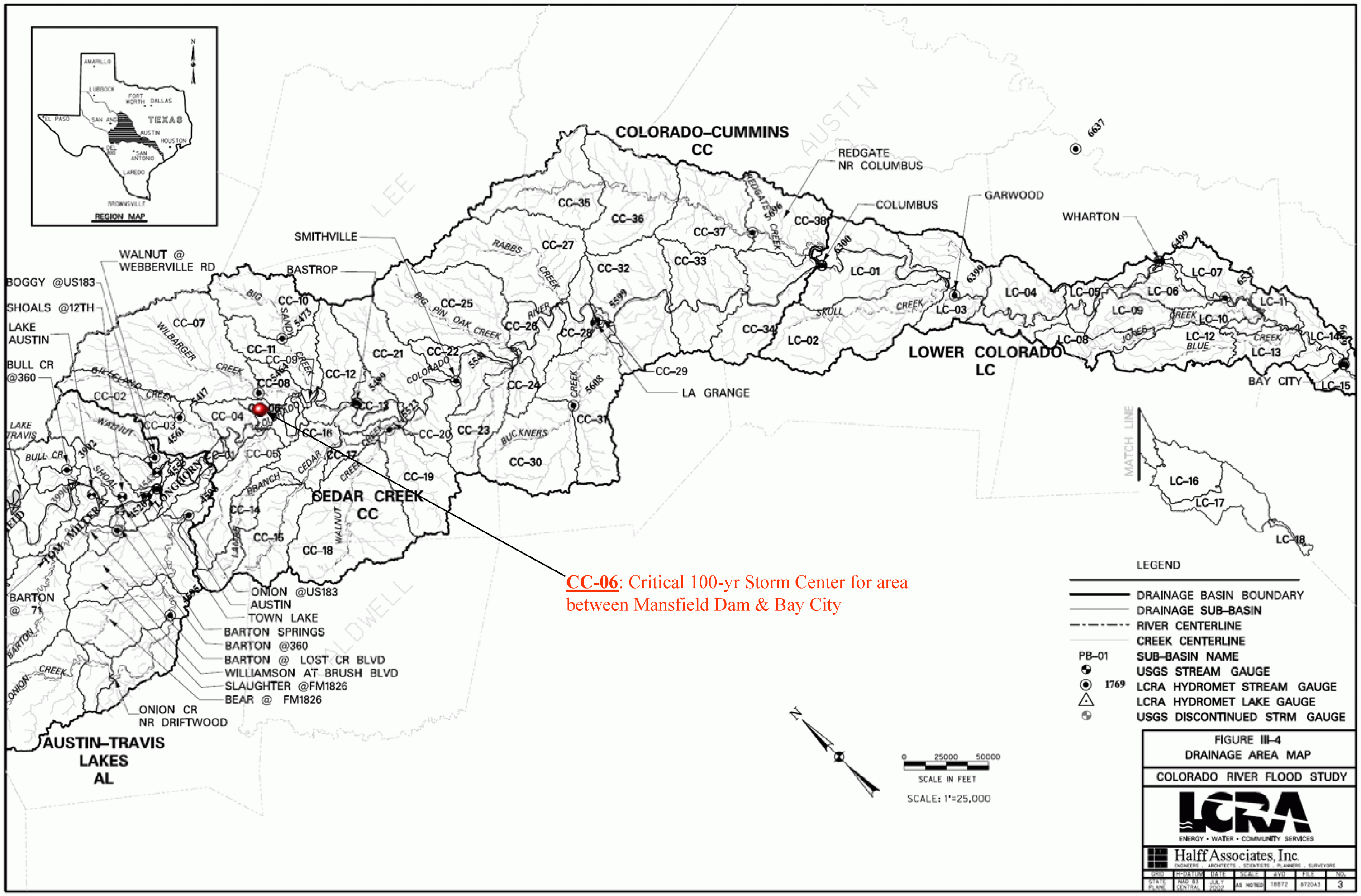

Figure 2.4S.3-2(a) Drainage Delineation of Subbasins between Mansfield Dam and Matagoda Bay (Modified from Reference 2.4S.3-8)

Figure 2.4S.3-2a Drainage Delineation of Subbasins between Mansfield Dam and Matagoda Bay (Modified from Reference 2.4S.3-8)

Figure 2.4S.3-2(b) Drainage Delineation of Subbasins between Mansfield Dam and Matagoda Bay (Modified from Reference 2.4S.3-8)

Figure 2.4S.3-2b Drainage Delineation of Subbasins between Mansfield Dam and Matagoda Bay (Modified from Reference 2.4S.3-8)

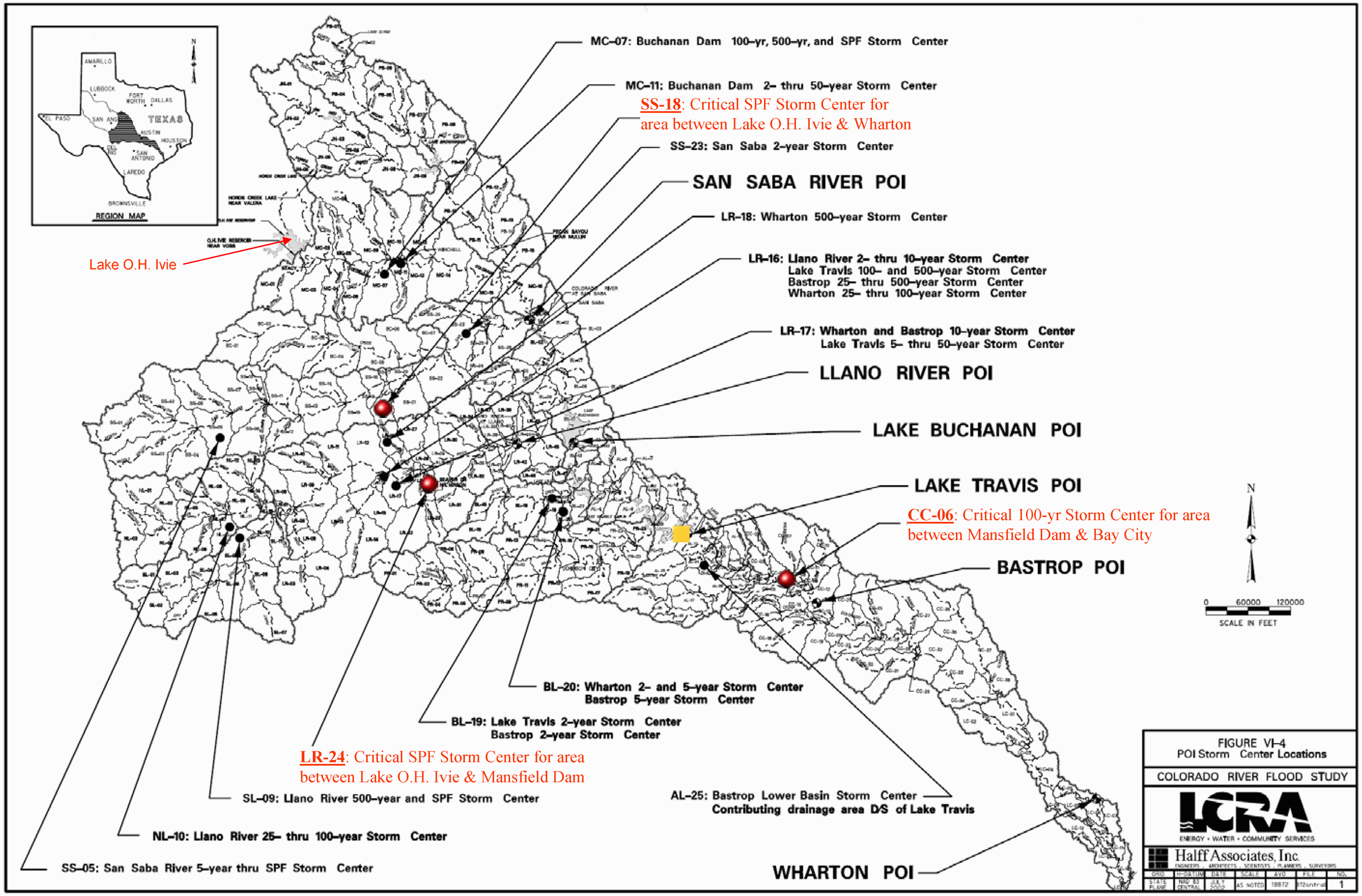

Figure 2.4S.3-3 Lower Colorado River Basin from Lake O.H. Ivie to Matagorda Bay (Modified from Reference 2.4S.3-8)

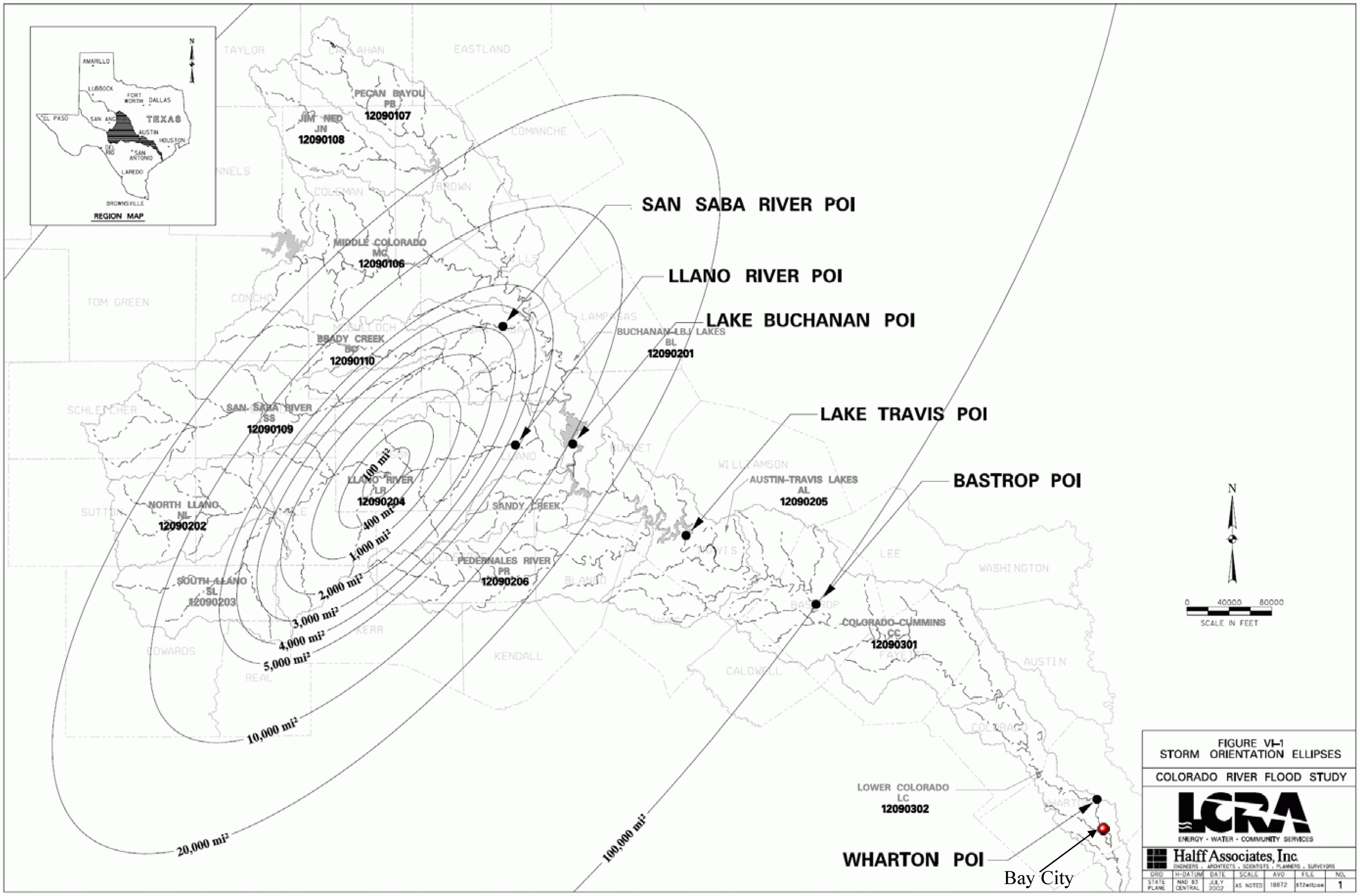

Figure 2.4S.3-4 Storm Orientation Pattern for Locations Upstream of Mansfield Dam (Modified from Reference 2.4S.3-8)

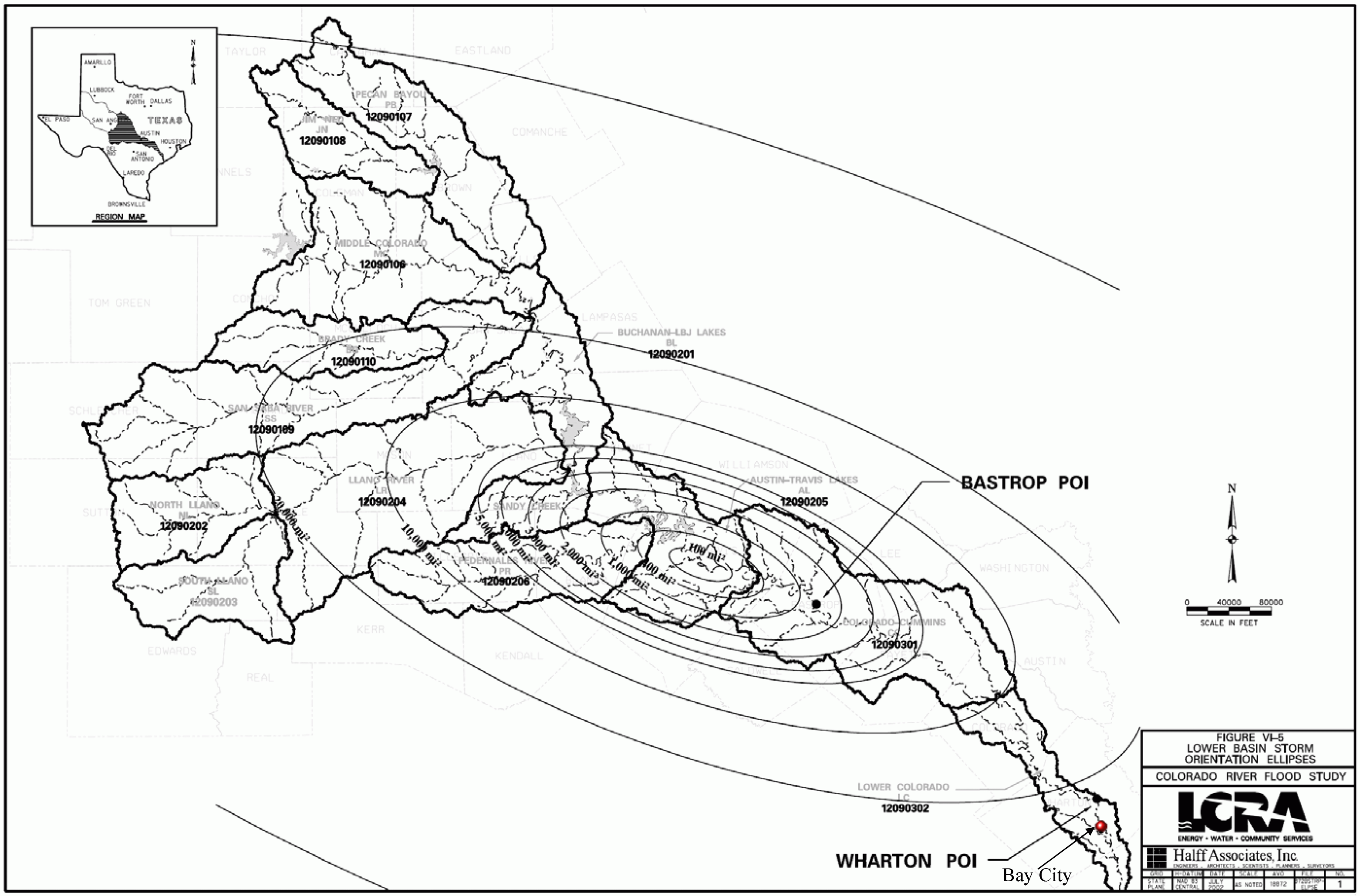

Figure 2.4S.3-5 Storm Orientation Pattern for Locations Downstream of Mansfield Dam (Modified from Reference 2.4S.3-8)

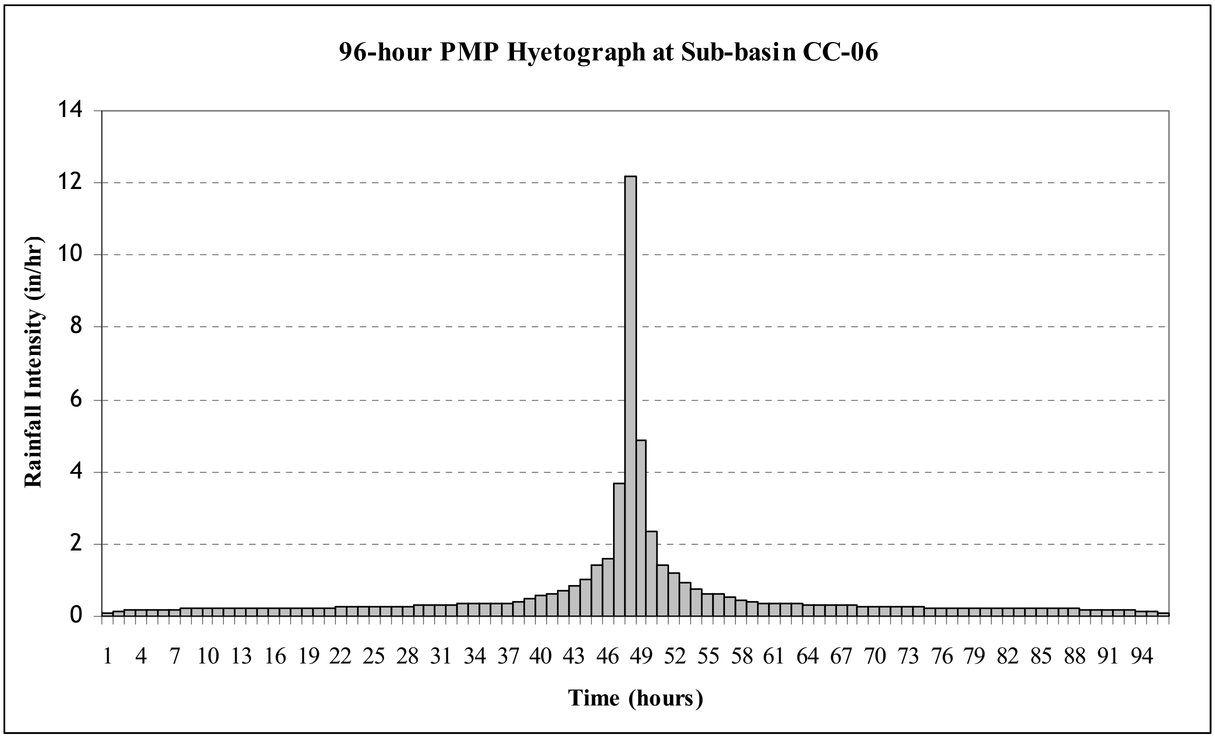

Figure 2.4S.3-6 96-hour PMP Hyetograph for Subbasin CC-06

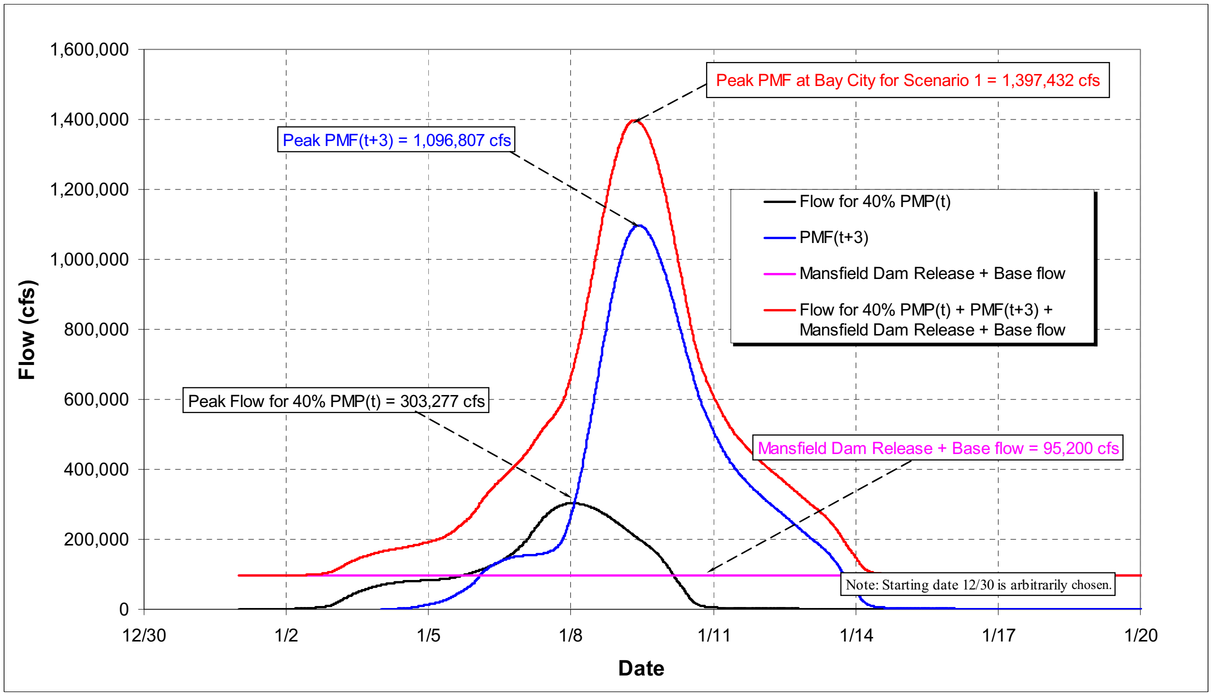

Figure 2.4S.3-7 PMF Hydrograph at Bay City for Scenario 1

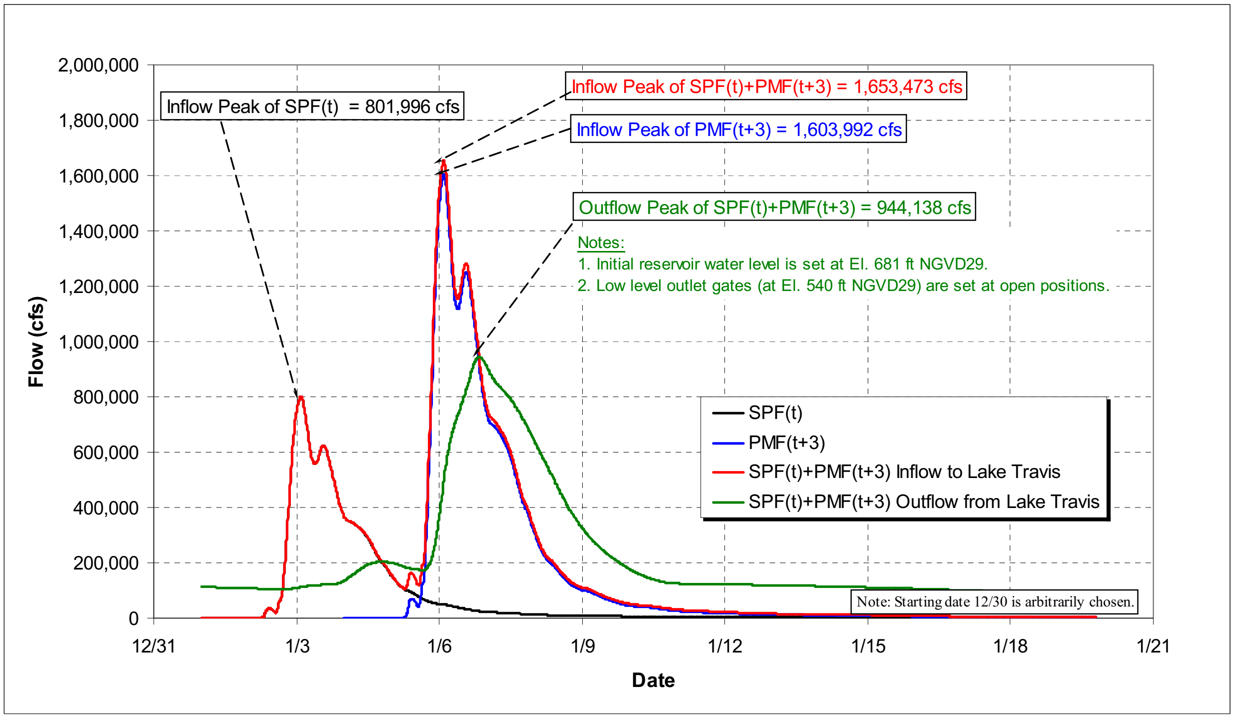

Figure 2.4S.3-8 Development of PMF Outflow Hydrograph at Lake Travis for Scenario 2

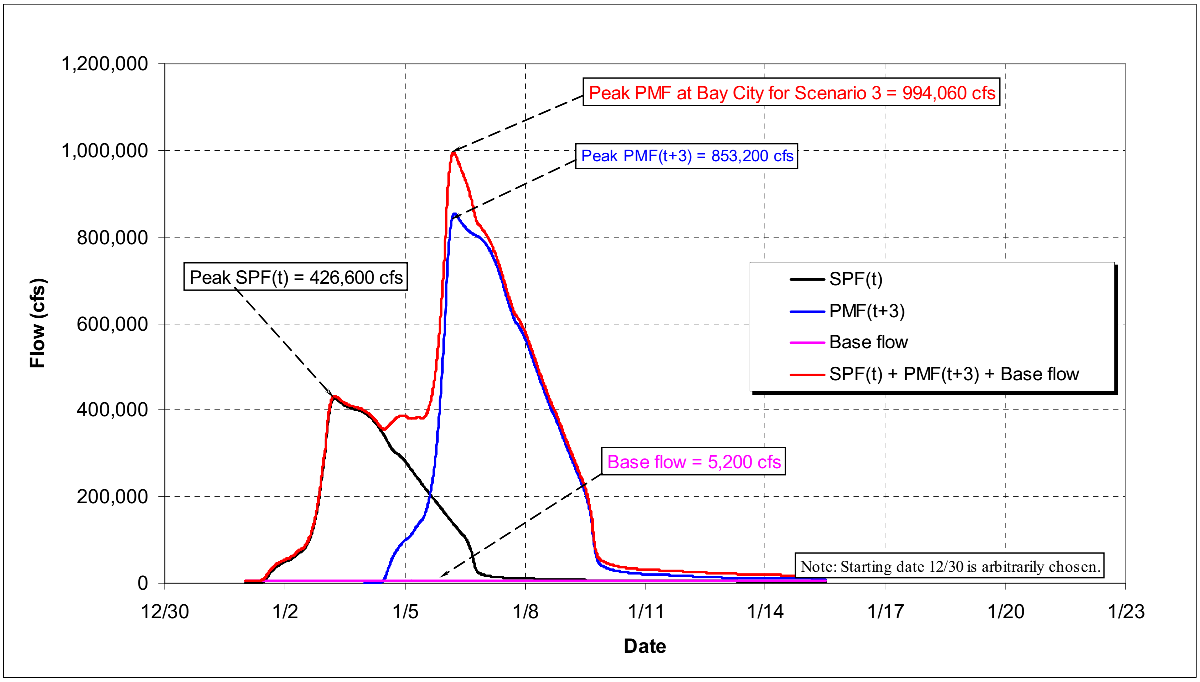

Figure 2.4S.3-9 PMF Hydrograph at Bay City for Scenario 3

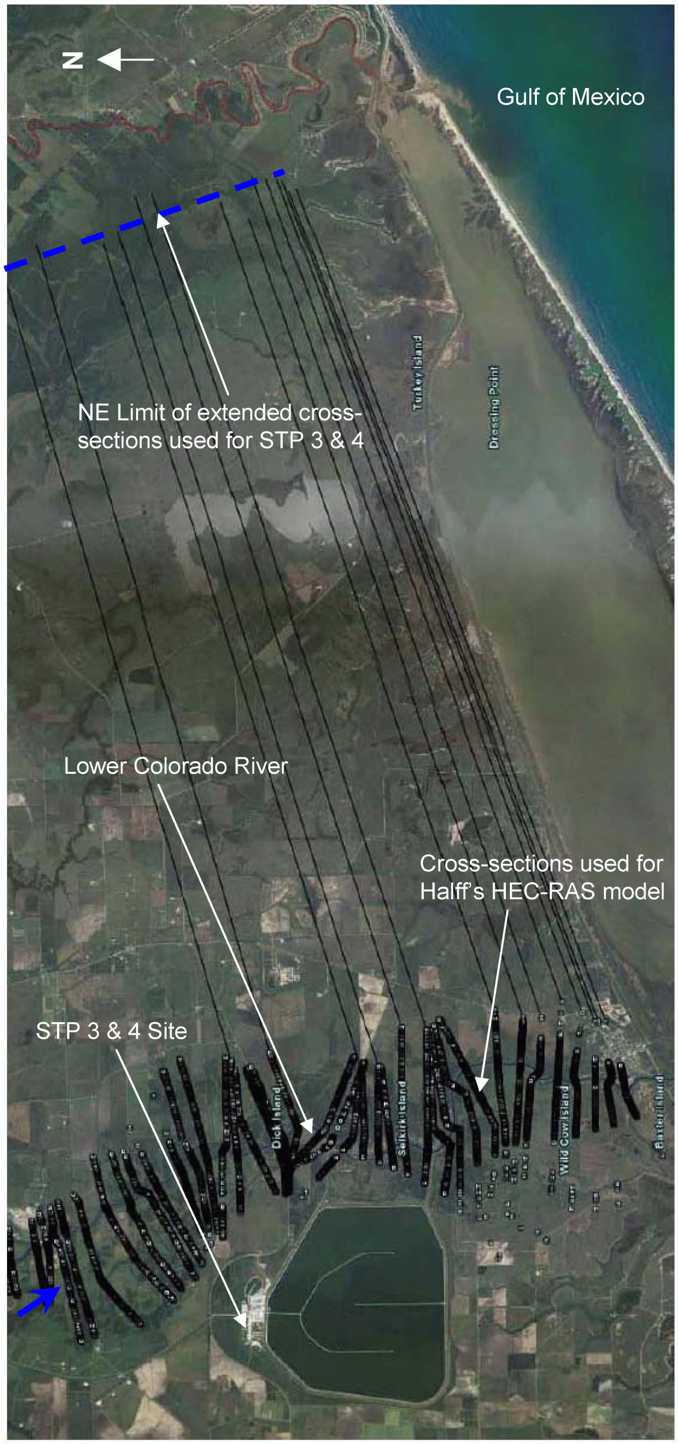

Figure 2.4S.3-10 Extended Cross sections – Most downstream section to STP 3 & 4 Site

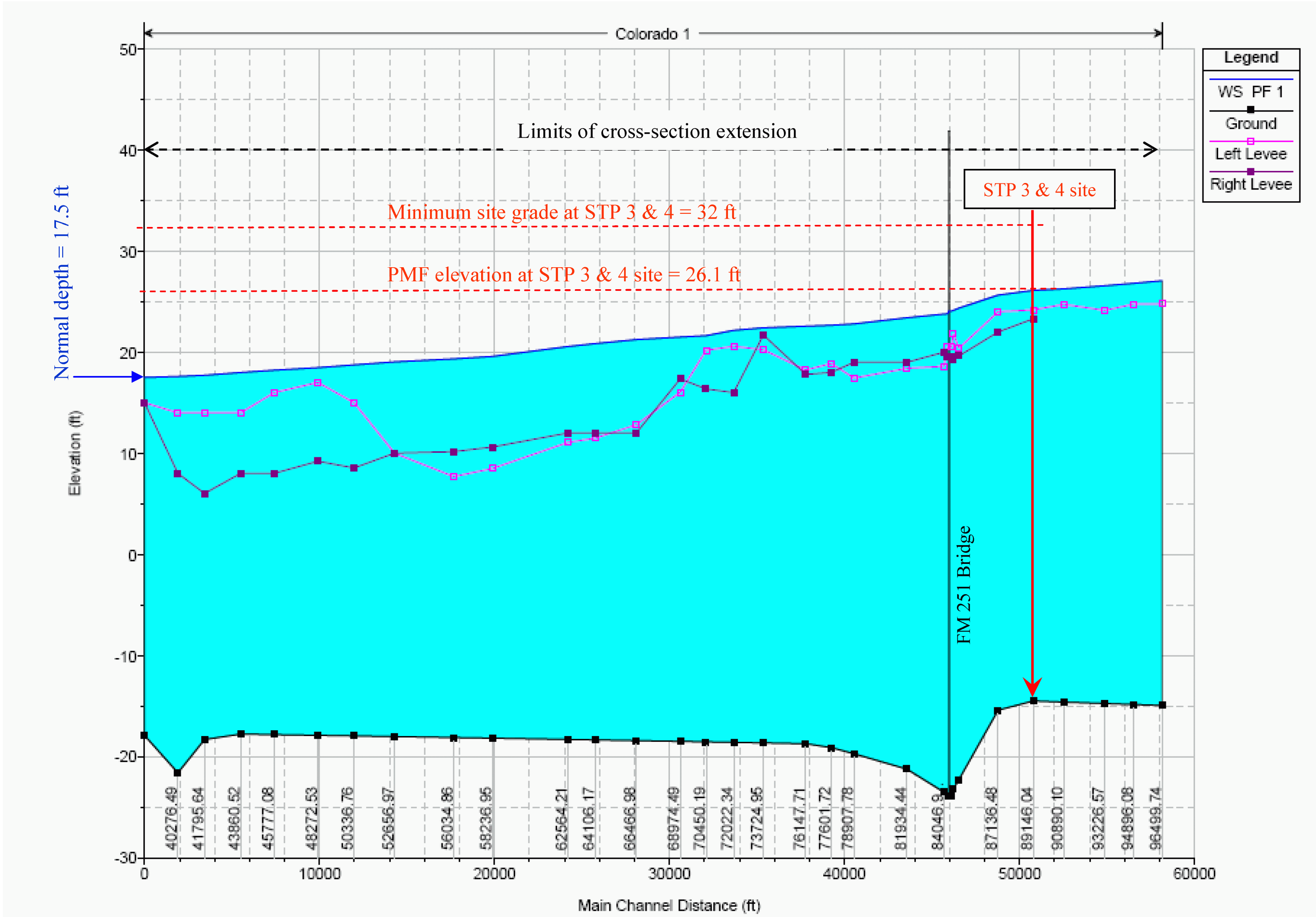

Figure 2.4S.3-11 PMF Elevation at STP 3 & 4 Site for Normal Depth Boundary Condition (Manning's *n* values equal to 1.2 times those used in the Halff model)

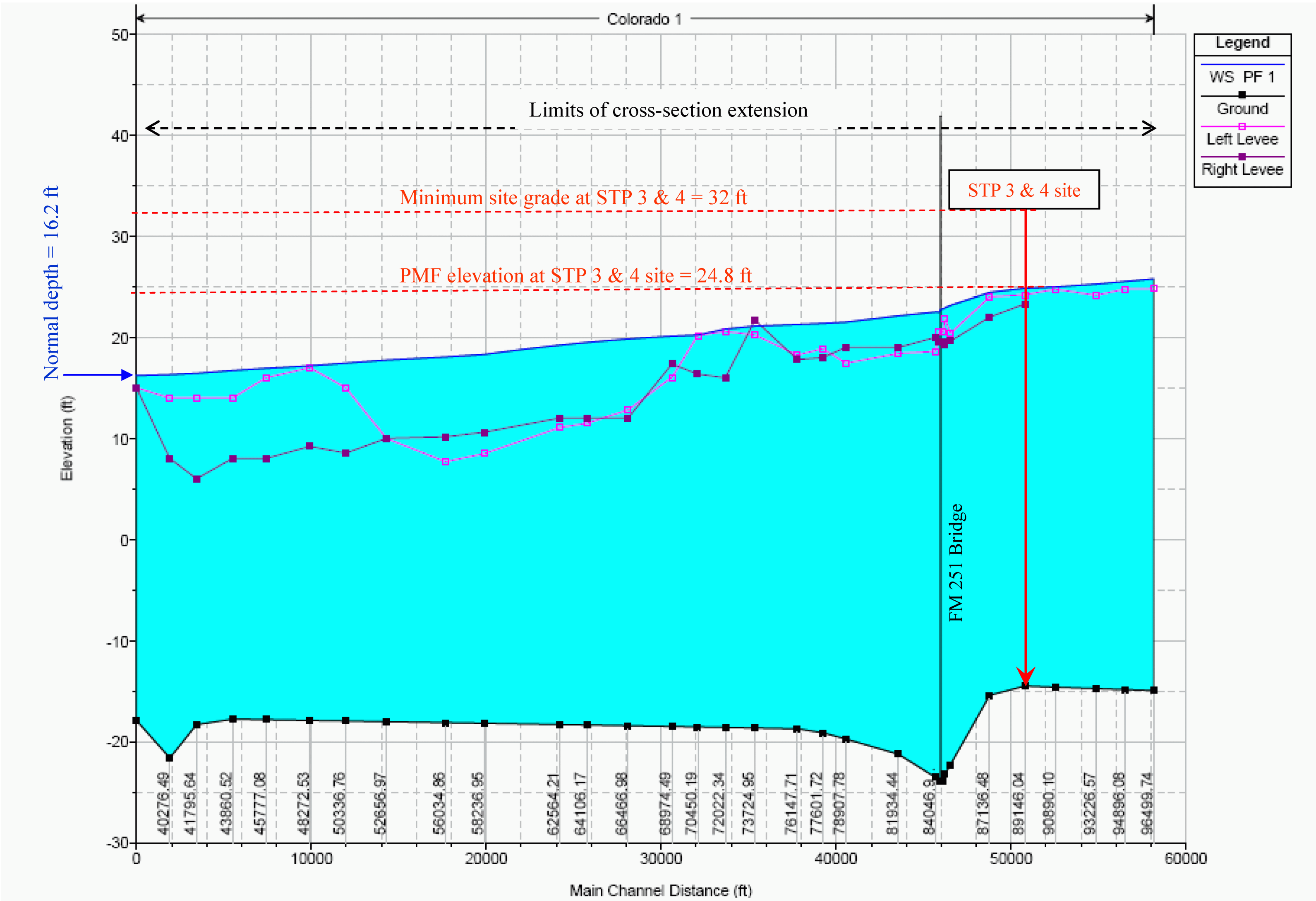

Figure 2.4S.3-12 PMF Elevation at STP 3 & 4 Site for Normal Depth Boundary Condition (Manning's *n* values equal to those used in the Halff model)

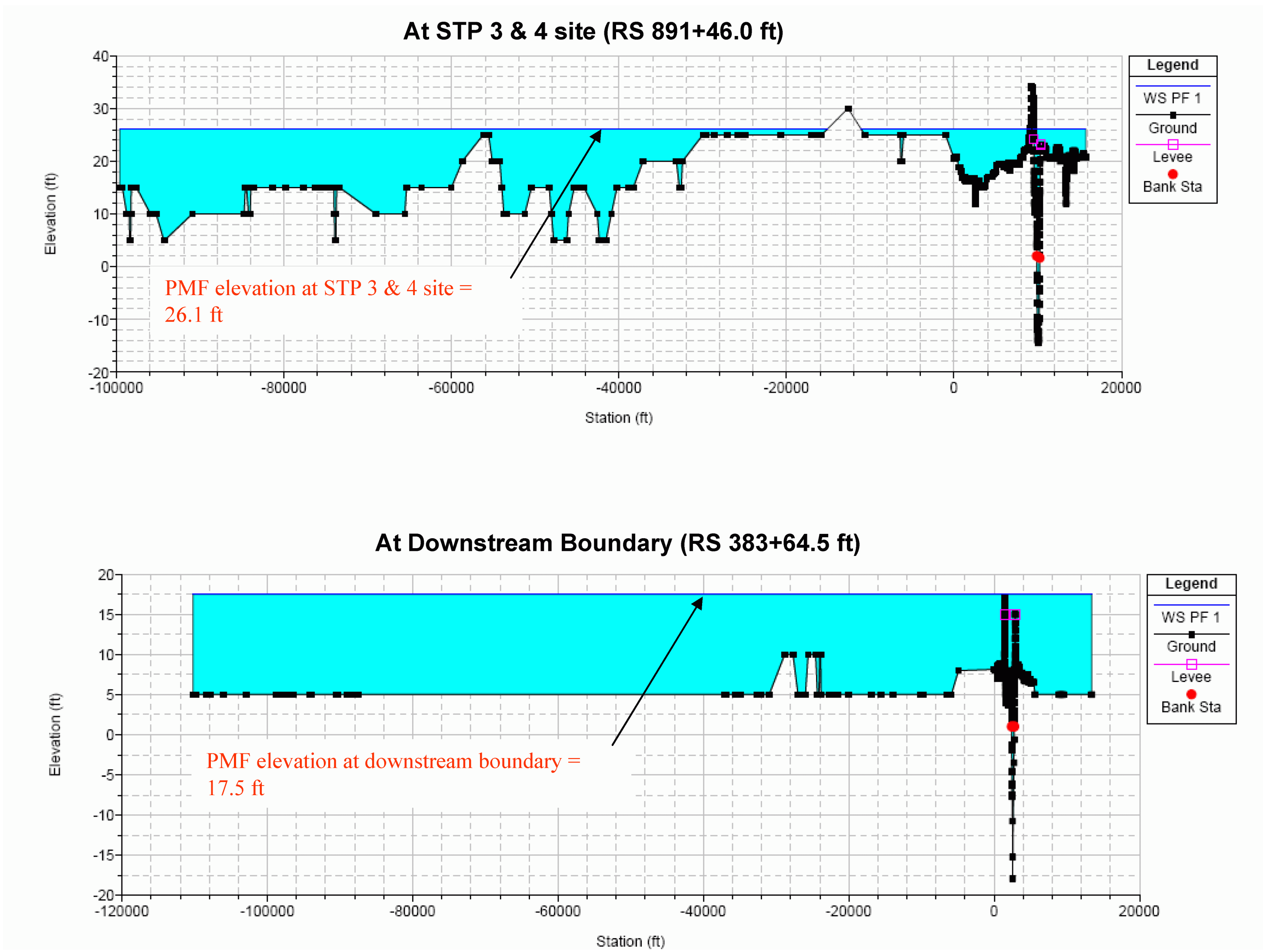

Figure 2.4S.3-13 PMF Water Levels at STP 3 & 4 Site (RS 891+46.0) and at Downstream Boundary (RS 383+64.5)
(Manning's *n* values equal to 1.2 times those used in the Halff model)

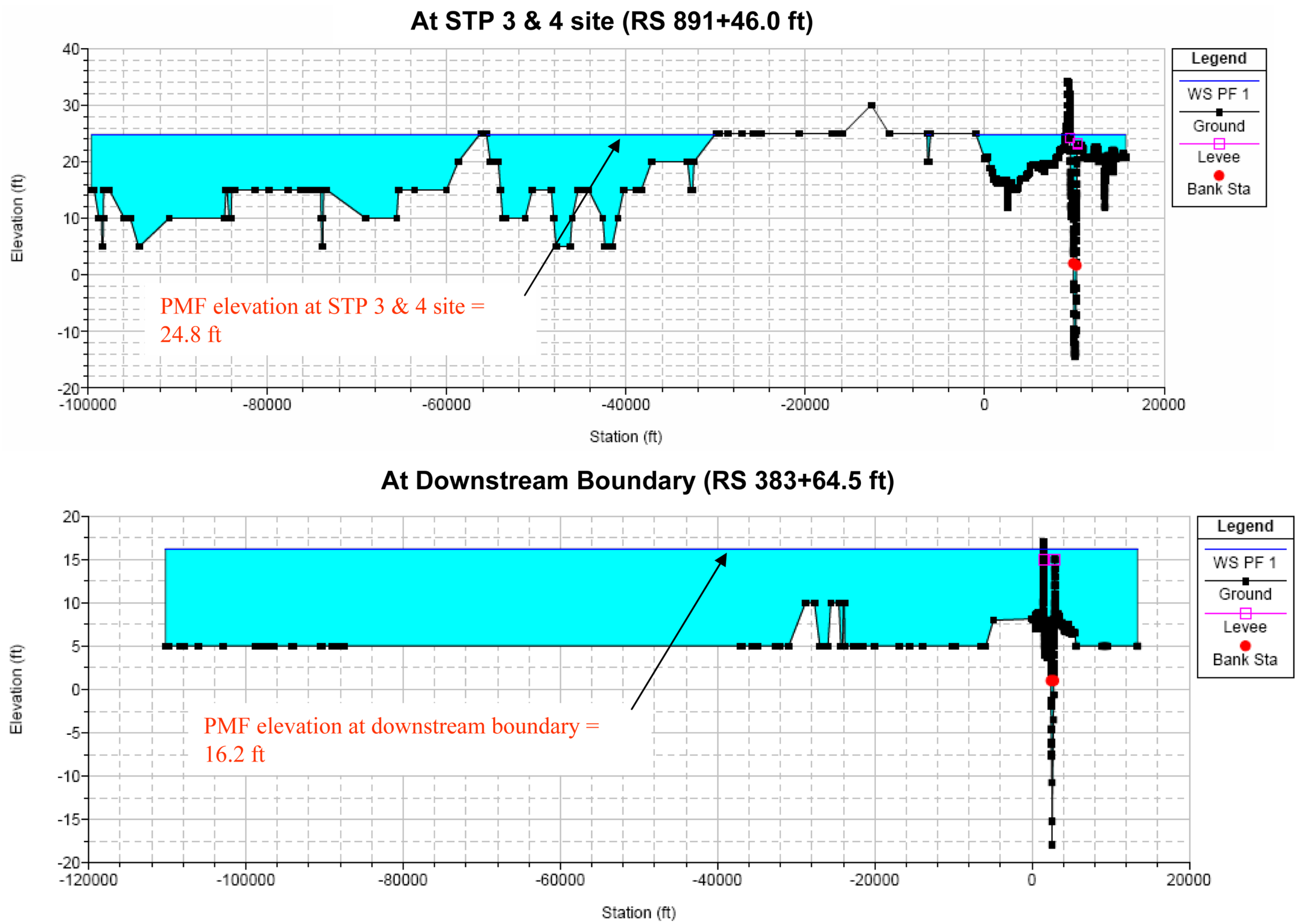

Figure 2.4S.3-14 PMF Water Levels at STP 3 & 4 Site (RS 891+46.0) and at Downstream Boundary (RS 383+64.5) (Manning's *n* values equal to those used in the Halff model)