UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

September 30, 2014

Mr. Dennis L. Koehl
President and CEO/CNO
STP Nuclear Operating Company
South Texas Project
South Texas Project Electric Generating Station
P.O. Box 289
Wadsworth, TX 77483

SUBJECT: SOUTH TEXAS PROJECT, UNITS 1 AND 2 – STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST FLOOD-CAUSING MECHANISM REEVALUATION (TAC NOS. MF1110 AND MF1111)

Dear Mr. Koehl:

By letter dated March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued a request for information pursuant to Title 10 of the *Code of Federal Regulations*, Section 50.54(f) (hereafter referred to as the 50.54(f) letter). The request was issued as part of implementing lessons-learned from the accident at the Fukushima Dai-ichi nuclear power plant. Enclosure 2 to the 50.54(f) letter requested licensees to reevaluate flood-causing mechanisms using present-day methodologies and guidance.

By letter dated March 13, 2013, STP Nuclear Operating Company responded to this request for the South Texas Project, Units 1 and 2.

The NRC staff has reviewed the information provided and, as documented in the enclosed staff assessment, determined that you provided sufficient information in response to the 50.54(f) letter. This closes out the NRC's efforts associated with TAC Nos. MF1110 and MF1111. If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

Sincerely,

Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket Nos. 50-498 and 50-499

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report

cc w/encl: Distribution via Listserv

STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST

FLOOD-CAUSING MECHANISM REEVALUATION

BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO FLOODING HAZARD REEVALUATION REPORT

STP NUCLEAR OPERATING COMPANY

SOUTH TEXAS PROJECT, UNITS. 1 AND 2

DOCKET NOS. 50-498 AND 50-499

INTRODUCTION

By letter dated March 12, 2012 (NRC, 2012a), the U.S. Nuclear Regulatory Commission (NRC) issued a request for information to all power reactor licensees and holders of construction permits in active or deferred status, pursuant to Title 10 of *the Code of Federal Regulations* (10 CFR), Section 50.54(f) "Conditions of license" (hereafter referred to as the "50.54(f) letter"). The request was issued in connection with implementing lessons-learned from the 2011 accident at the Fukushima Dai-ichi nuclear power plant, as documented in the "Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident." (NRC, 2011b).[1] Recommendation 2.1 in that document recommended that the staff issue orders to all licensees to reevaluate seismic and flooding for their sites against current NRC requirements and guidance. Subsequent Staff Requirements Memoranda associated with Commission Papers SECY-11-0124 (NRC, 2011c) and SECY-11-0137 (NRC, 2011d), directed the NRC staff to issue requests for information to licensees pursuant to 10 CFR 50.54(f).

Enclosure 2 to the 50.54(f) letter requested that licensees reevaluate the flood hazard for their respective sites using present-day methods and regulatory guidance used by the NRC staff when reviewing applications for early site permits (ESPs) and combined licenses (COLs). The required response section of Enclosure 2 specified that NRC staff would provide a prioritization plan indicating Flooding Hazard Reevaluation Report (FHRR) deadlines for each plant. On May 11, 2012, the staff issued (NRC, 2012b) its prioritization of the FHRRs.

If the reevaluated hazard for all flood-causing mechanisms is not bounded by the current plant design-basis flood hazard, an Integrated Assessment will be necessary. The FHRR and the responses to the associated Requests for Additional Information (RAIs) will provide the hazard input necessary to complete the Integrated Assessment report, as described in Japan Lessons-Learned Project Directorate (JLD) interim staff guidance(ISG) JLD-ISG-2012-05, "Guidance for Performing the Integrated Assessment for External Flooding." (NRC, 2012c)

By letter dated March 13, 2013, STP Nuclear Operating Company (STPNOC, the licensee) provided its FHRR for South Texas Project (STP, South Texas), Units 1 and 2. The licensee did not identify any interim actions.

---

[1] Issued as an enclosure to Commission Paper SECY-11-0093 (NRC, 2011a).

1.0 REGULATORY BACKGROUND

1.1 Applicable Regulatory Requirements

This section describes present-day regulatory requirements that are applicable to the FHRR.

Section 50.34(a)(1), (a)(3), (a)(4), (b)(1), (b)(2), and (b)(4), of 10 CFR, describes the required content of the preliminary and final safety analysis reports, including a discussion of the facility site with a particular emphasis on the site evaluation factors identified in 10 CFR Part 100. The licensee should provide any pertinent information identified or developed since the submittal of the preliminary safety analysis report in the final safety analysis report.

Section 50.54(f) of 10 CFR states that a licensee shall at any time before expiration of its license, upon request of the Commission, submit written statements, signed under oath or affirmation, to enable the Commission to determine whether or not the license should be modified, suspended, or revoked. The 50.54(f) letter requested licensees reevaluate the flood-causing mechanisms for their respective sites using present-day methodologies and regulatory guidance used by the NRC for the ESP and COL reviews.

General Design Criterion 2 in Appendix A of Part 50 states that structures, systems, and components (SSCs) important to safety at nuclear power plants must be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunamis, and seiches, without loss of capability to perform their intended safety functions. The design bases for these SSCs are to reflect appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area. The design bases are also to have sufficient margin to account for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Section 50.2 of 10 CFR defines the design-basis as the information that identifies the specific functions that an SSC of a facility must perform, and the specific values or ranges of values chosen for controlling parameters as reference bounds for design which each licensee is required to develop and maintain. These values may be (a) restraints derived from generally accepted "state of the art" practices for achieving functional goals, or (b) requirements derived from an analysis (based on calculation or experiments or both) of the effects of a postulated accident for which an SSC must meet its functional goals.

Section 54.3 of 10 CFR defines the "current licensing basis" (CLB) as: "the set of NRC requirements applicable to a specific plant and a licensee's written commitments for ensuring compliance with and operation within applicable NRC requirements and the plant-specific design-basis (including all modifications and additions to such commitments over the life of the license) that are docketed and in effect." This includes 10 CFR Parts 2, 19, 20, 21, 26, 30, 40, 50, 51, 52, 54, 55, 70, 72, 73, 100 and appendices thereto; orders; license conditions; exemptions; and technical specifications as well as the plant-specific design-basis information as documented in the most recent final safety analysis report. The licensee's commitments made in docketed licensing correspondence, which remain in effect, are also considered part of the CLB.

Present-day regulations for reactor site criteria (Subpart B to 10 CFR Part 100 for applications on or after January 10, 1997) state, in part, that the physical characteristics of the site must be evaluated and site parameters established such that potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site.

Factors to be considered when evaluating sites include the nature and proximity of dams and other man-related hazards (10 CFR 100.20(b)) and the physical characteristics of the site, including the hydrology (10 CFR 100.21(d)).

1.2 Enclosure 2 to the 50.54(f) Letter

The 50.54(f) letter requests all power reactor licensees and construction permit holders reevaluate all external flooding-causing mechanisms at each site. The reevaluation should apply present-day methods and regulatory guidance that are used by the NRC staff to conduct ESP and COL reviews. This includes current techniques, software, and methods used in present-day standard engineering practice. If the reevaluated flood-causing mechanisms are not bounded by the current plant design-basis flood hazard, an Integrated Assessment will be necessary.

1.2.1 Flood-Causing Mechanisms

Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) discusses flood-causing mechanisms for the licensee to address in the FHRR. Table 2.2-1 lists the flood-causing mechanisms the licensee should consider. Table 2.2-1 also lists the corresponding *Standard Review Plan* (NRC, 2007) sections and applicable interim staff guidance containing acceptance criteria and review procedures. The licensee should incorporate and report associated effects per JLD-ISG-2012-05, (NRC, 2012c) in addition to the maximum water level associated with each flood-causing mechanism.

1.2.2 Associated Effects

In reevaluating the flood-causing mechanisms, the "flood height and associated effects" should be considered. The ISG for performing the integrated assessment for external flooding, JLD-ISG-2012-05 (NRC, 2012c), defines "flood height and associated effects" as the maximum stillwater surface elevation plus:

- wind waves and run-up effects
- hydrodynamic loading, including debris
- effects caused by sediment deposition and erosion
- concurrent site conditions, including adverse weather conditions
- groundwater ingress
- other pertinent factors

1.2.3 Combined Effects Flood

The worst flooding at a site that may result from a reasonable combination of individual flooding mechanisms is sometimes referred to as a "Combined Effects Flood." Even if some or all of these individual flood-causing mechanisms are less severe than their worst-case occurrence, their combination may still exceed the most severe flooding effects from the worst-case occurrence of any single mechanism in the 50.54(f) letter (See the Standard Review Plan (SRP), Section 2.4.2, Area of Review 9 (NRC, 2007)). Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) describes the "Combined Effect Flood" [2] as defined in standard ANSI/ANS 2.8-1992 (ANS, 1992) as follows:

> *For flood hazard associated with combined events, American Nuclear Society (ANS) 2.8-1992 provides guidance for combination of flood causing mechanisms for flood hazard at nuclear power reactor sites. In addition to those listed in the ANS guidance, additional plausible combined events should be considered on a site specific basis and should be based on the impacts of other flood causing mechanisms and the location of the site.*

If two less severe mechanisms are plausibly combined (per ANSI/ANS-2.8-1992 (ANS, 1992) and SRP, Section 2.4.2, Areas of Review 9 (NRC, 2007)), then the staff will document and report the result as part of one of the hazard sections. An example of a situation where this may occur is flooding at a riverine site located where the river enters the ocean. For this site, storm surge and river flooding should be plausibly combined.

1.2.4 Flood Event Duration

Flood event duration was defined in the ISG for the integrated assessment for external flooding, JLD-ISG-2012-05 (NRC, 2012c), as the length of time during which the flood event affects the site. It begins when conditions are met for entry into a flood procedure, or with notification of an impending flood (e.g., a flood forecast or notification of dam failure), and includes preparation for the flood. It continues during the period of inundation, and ends when water recedes from the site and the plant reaches a safe and stable state that can be maintained indefinitely. Figure 2.2.4-1 illustrates flood event duration.

1.2.5 Actions Following the FHRR

For the sites where the reevaluated flood hazard is not bounded by the current design-basis flood hazard for all flood-causing mechanisms, the 50.54(f) letter requests licensees and construction permit holders to

- Submit an Interim Action Plan with the FHRR documenting actions planned or already taken to address the reevaluated hazard
- Perform an Integrated Assessment subsequent to the FHRR to (a) evaluate the effectiveness of the current licensing basis (i.e., flood protection and mitigation systems), (b) identify plant-specific vulnerabilities, and (c) assess the effectiveness of existing or planned systems and procedures for protecting against and mitigating consequences of flooding for the flood event duration

If the reevaluated flood hazard is bounded by the current design-basis flood hazard for all flood-causing mechanisms at the site, licensees are not required to perform an Integrated Assessment at this time.

---

[2] For the purposes of this Staff Assessment, the terms "combined effects" and "combined events" are synonyms.

2.0 TECHNICAL EVALUATION

The NRC staff has reviewed the information provided for the flood hazard reevaluation of STP, Units 1 and 2. The licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews. The staff's review and evaluation is provided below.

The licensee's flood hazard reevaluation studies were conducted partly using conventional units of measure and partly using metric units. This staff assessment presents conventional units followed by the equivalent in metric units, in parentheses. Original units used in measurements can be found in the corresponding referenced documents. Because unit conversions may cause loss of precision, the original units are definitive.

The site grade at the powerblock is elevation 28.00 feet (ft) (8.53 m) on the National Geodetic Vertical Datum of 1929 (NGVD29). Unless stated otherwise, all elevations in this staff assessment are given with respect to NGVD29. Table 3.0-1 provides the summary of controlling reevaluated flood-causing mechanisms, including associated effects, the licensee computed to be higher than the powerblock elevation.

2.1 Site Information

The 50.54(f) letter includes the SSCs important to safety, and the ultimate heat sink (UHS), in the scope of the hazard reevaluation. Per the 50.54(f) letter, Enclosure 2, Requested Information, Hazard Reevaluation Report, Item a, the licensee included pertinent data concerning these SSCs in the FHRR.

The 50.54(f) letter, Enclosure 2 (Recommendation 2.1: Flooding), Requested Information, Hazard Reevaluation Report, Item a, describes site information to be contained in the FHRR. The staff reviewed and summarized this information as follows.

2.1.1 Detailed Site Information

STP, Units 1 and 2 are located in south-central Matagorda County, just west of the Colorado River, and 8 mi (13 km) north-north-west of Matagorda, Texas. Two generating units (Units 1 and 2) occupy the site. Both are pressurized-water reactors (PWRs) with turbine generators. The units are located 600 ft (180 m) apart. Each unit is rated for 3,853 Megawatt thermal core thermal power.

The major hydrologic features located off-site are the Gulf of Mexico, located 16.9 mi (27.2 km) to the south; the Colorado River, located just to the east of the plant; and Little Robbins Slough, located 9 mi (15 km) northwest of Matagorda.

The STP site has mainly flat topography with few gentle slopes. Elevations across the site range from 15 ft (4.6 m) NGVD29 to 30 ft (9.1 m) NGVD29 with plant grade of 28 ft (8.53 m) NGVD29.

The UHS, also referred to as the Essential Cooling Pond (ECP), contains enough water to supply the Essential Cooling Water System (ECWS) for 30 days. The normal operating range for the ECP is from 25.6 ft (7.80 m) NGVD29 to 26 ft (7.92 m) NGVD29.

The largest on-site feature is the Main Cooling Reservoir (MCR). The MCR has a surface area of 7,000 acres (28 km^2) and a typical maximum operating level of 49 ft (14.9 m) NGVD29. The MCR is not safety related.

2.1.2 Design-Basis Flood Hazards

The current design-basis flood levels are summarized by flood-causing mechanism in Table 3.1.2-1.

2.1.3 Flood-Related Changes to the Licensing Basis

No changes have been made to the licensing basis.

2.1.4 Changes to the Watershed and Local Area

There have been no large dams or impoundments proposed or constructed on the Colorado River or its tributaries since the operation of STP, Units 1 and 2.

Local changes include the construction of a vehicle barrier for security purposes. Its effect was incorporated in the reevaluation of local intense precipitation.

2.1.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

Floods that could result from all causes and/or combinations thereof were analyzed for the Colorado River, coastal area, local site drainage, upstream dams and the MCR. The critical flood levels at STP Units 1 and 2 determined from these analyses, result from a postulated breach of a portion of the north embankment of the MCR which determine controlling levels for the power block and the essential cooling water intake structure (ECWIS).

The current licensing basis provides flood protection and mitigation to an elevation of 50.8 ft (15.5 m). The flood protection and mitigation features were designed using the following assumptions and inputs: the SSCs are designed to withstand the maximum flood level and associated effects and remain functional; or the SSCs are housed within seismic Category 1 structures, which are designed to withstand the maximum flood level and associated effects and remain functional.

The buildings that house SSCs at STP, Units 1 and 2 are equipped with watertight panels, watertight access covers/doors, waterstops on construction joints and slabs, and other features to prevent flooding in safety-related areas. In addition, the seismic Category I buildings have walls and surface slabs that are waterproofed below grade to protect against potential flooding from groundwater. The Fuel Handling Building has sumps equipped with pumps to handle any infiltrated groundwater.

2.1.6 Additional Site Details to Assess the Flood Hazard

Flooding events at STP, Units 1 and 2 are postulated to occur in any plant operational mode, and thus no specific modes are identified for protection and mitigation from flooding.

2.1.7 Plant Walkdown Activities

Requested Information Item 1.c and Attachment 1 to Enclosure 2, Step 6, in the 50.54(f) letter requires licensees to report any relevant information from the results of the plant walkdown activities associated with Enclosure 4 of the 50.54(f) letter.

The licensee responded, by letter dated June 5, 2012, that they would perform the plant walkdown activities (STPNOC, 2012c).

By letter dated November 26, 2012, STPNOC provided the flood walkdown report for South Texas Project, Units 1 and 2 (STPNOC, 2012b).

The staff prepared a staff assessment report, dated June 23, 2014 (NRC, 2014), to document its review of the walkdown report. The staff concluded that the licensee's implementation of flooding walkdown methodology met the intent of the walkdown guidance..

2.2 Local Intense Precipitation and Associated Site Drainage

The licensee reported in the FHRR that the reevaluated probable maximum flood elevation, including associated effects, for local intense precipitation (LIP) is based on a stillwater-surface elevation of 33.0 ft (10.1 m) NGVD29 within the power block area (STPNOC, 2013). This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis probable maximum flood elevation for the LIP and associated site drainage hazard is a stillwater-surface elevation of 32.0 ft (9.8 m) NGVD29 (FHRR, Table 1.2-1).

To supplement the FHRR, the staff issued an RAI to the licensee. The licensee's response (STPNOC, 2014a, b) provided copies of model input files; described sources of elevation data; and described the basis for classifying probable maximum flood flow as shallow concentrated flow.

The staff reviewed the LIP and associated site drainage analysis, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance.

The licensee's reevaluation estimated hazards from LIP and site drainage in 14 subbasins (see Figure 3.2-1). Flooding concerns are associated with the capacity for site grade to route rainwater away from SSCs.

Site topography suggests that precipitation will drain either southeast to Kelly Lake or southwest through Little Robbins Slough (see Table 3.2-1). The licensee characterized two drainage paths.

- The first drainage path includes a storm-drainage network consisting of catch basins, pipes, and ditches that collect runoff from the power block area; this network routes flow eastward to Kelly Lake.
- The site area west of North Access Road drains westward to Little Robbins Slough through the second drainage path.

At a sufficient depth, runoff can flow into SSCs. The licensee identified the following STP, Units 1 and 2 safety-related SSCs:

- Reactor Containment Building
- Fuel-Handling Building
- Mechanical-Electrical Auxiliaries Building
- Isolation Valve Cubicle
- Diesel-Generator Building
- Auxiliary Feedwater Storage Tank
- ECWIS

- Essential Cooling Water Discharge Structure, and the ECP except for its north embankment

The minimum current design-basis flood (DBF) water-surface elevation for these SSCs is 13.6 m (44.5 ft) NGVD29.

2.2.1 Local Intense Precipitation

The licensee stated that the methods it used in the flood reevaluation were the same as those used in the STP, Units 3 and 4 combined license application (COLA). These methods are consistent with present-day regulatory guidance including NUREG/CR-7046 (NRC, 2011e). The staff reviewed the STP, Units 3 and 4 COLA based on guidance in Section 2.4 of the SRP (NRC, 2007). The guidance for flood estimation in the SRP (NRC, 2007) is based on the methods described in Regulatory Guide 1.59 (NRC, 1977), supplemented by current engineering practices described in NUREG/CR-7046 (NRC, 2011e). Therefore, the staff agrees that the licensee has used flood estimation methods for the LIP event that are generally consistent with present-day practices.

Table 3.2-2 provides the local probable maximum precipitation (PMP) depths estimated by the licensee. During its review, the staff compared the estimated PMP depths with estimates and measurements of precipitation depths presented previously.

The STP, Units 1 and 2, Updated Final Safety Analysis Report, Revision 15 (UFSAR) (STPNOC, 2010), states that the all-season 10-mi^2 (26-km^2), 6-h duration rainfall was 32.5 in (82.6 cm) for the site. This value is slightly larger than the value of 32.0 in (81.3 cm) provided in FHRR Table 2.1-1.

The staff also compared the PMP depths from the FHRR with the values reported previously in the STP, Units 3 and 4 Combined License Application, Final Safety Analysis Report, Revision 7 (FSAR) (STPNOC, 2012a). The PMP depths are identical, except for the 10-mi^2 (26-km^2), 12-h event (see Table 3.2-2). The FHRR value is 0.9 in (2 cm) larger than that in the FSAR. The proposed STP, Units 3 and 4 are adjacent to existing STP, Units 1 and 2. The staff determined that neither the present standard methods nor values associated with estimating the PMP have changed since the staff's review of the STP, Units 3 and 4 COLA.

The licensee stated in Section 2.1.1 of the FHRR that a review of historical precipitation records for the region since the publication of National Oceanic and Atmospheric Administration (NOAA) Hydrometeorological Reports (HMRs) 51 (Schreiner and Riedel, 1978) and 52 (Hansen, Schreiner, and Miller, 1982) identified no events exceeding the PMP. The licensee noted in the FHRR that the 47.1 in (120 cm) PMP depth for the 10-mi^2 (26-km^2), 24-h event derived from HMR 51 bounds the historical record 24-h event total of 43 in (109 cm) observed at Alvin, Texas on July 25 through 26, 1979 (STPNOC, 2013).

The staff found that the NOAA State Climate Extremes Committee website (NOAA, 2013) reported a 24-h point precipitation of 42 in (107 cm) at Alvin, Texas, for the same storm on July 25 through 26, 1979. The value of 42 in (107 cm) is the greatest 24-h point rainfall actually observed. The value of 43 in (109 cm) cited by the licensee "appears to have been estimated in a post-storm survey" (NOAA, 2013).

In conclusion, the PMP values presented in the FHRR and shown in Table 3.2-2 appear to be reasonable due to the similarities of licensee-reported local PMP depths in the FHRR and those reported in STP, Units 3 and 4 FSAR, the STP,Units 3 and 4 site proximity to STP, Units 1 and 2, lack of changes of methods to estimate PMP, and historical record precipitation in Texas is bounded by the PMP estimated from the HMRs.

2.2.2 Runoff Analyses

The licensee stated that the methods it used in the flood reevaluation were the same as those used in the STP, Units 3 and 4 COLA. These methods are consistent with present-day regulatory guidance and NUREG/CR-7046 (NRC, 2011e). The staff reviewed the STP, Units 3 and 4 COLA based on guidance in Section 2.4 of the Standard Review Plan (NRC,2007). The staff guidance for flood estimation in the SRP (NRC, 2007) is based on the methods described in Regulatory Guide 1.59 (NRC, 1977), supplemented by current engineering practices described in NUREG/CR-7046.

The licensee estimated the probable maximum flood (PMF) from a LIP event on the STP, Units 1 and 2 site (STPNOC, 2013). To estimate runoff and perform hydrologic routing, the licensee used the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) software (USACE, 2010a). The PMP depths shown in Table 3.2-2 were temporally distributed using the HEC-HMS Meteorological Model module (STPNOC, 2013). The licensee used the PMP depths for all durations and the HEC-HMS frequency storm option of the Meteorological Model to develop a 6-h PMP storm event. The licensee constructed a 6-h hyetograph with a total precipitation depth of 31.68 in (80.5 cm), which is slightly less than the 6-h PMP of 32 in (81.3 cm).

The licensee used the HEC-HMS software to evaluate runoff and hydrologic routing for each of the 14 subbasins (STPNOC, 2013. Figure 3.2-1 shows the subbasins. Figure 3.2-2 shows the node-link (subbasin-junction) schematic of the subbasins as represented in the HEC-HMS analysis. The subbasins were assumed to be nearly impervious, with a runoff curve number of 98. The times of concentration for the subbasins were estimated using Natural Resource Conservation Service (NRCS) methods (NRCS, 1986). The licensee computed the times of concentration as a combination of sheet flow, shallow concentrated flow, and ditch flow; the times of concentration were reduced by 25 percent to account for nonlinearity in runoff processes during extreme flood condition. The reduced times of concentration are shown in FHRR Table 2.1-4 and reproduced in Table 3.2-3 of this staff assessment. The subbasin peak discharges were estimated using the HEC-HMS lag method option. The licensee computed the lag times for each subbasin as 60 percent of the respective, reduced time of concentration.

Table 3.2-4 summarizes the licensee's calculated peak discharges from the HEC-HMS analysis. The licensee found that the HEC-HMS runoff depth was 31.68 in (80.47 cm) (STPNOC, 2013, Table 2.1-5), which is consistent with the 6-h PMP depth of 32 in (81.3 cm) and a runoff curve number of 98. The licensee estimated that the peak discharge to Kelly Lake was about 24,900 ft^3/s (705 m^3/s) and the peak discharge to the Little Robbins Slough was about 5040 ft^3/s (143 m^3/s) (STPNOC, 2013).

Given the significant role that the HEC-HMS model performed in the licensee's analysis of the flood from local intense precipitation and the need to review the formulation of the model's complex spatially and temporally distributed input, the staff reviewed the HEC-HMS model input files provided by the licensee (STPNOC, 2014a). Based on its review, the staff determined that the licensee's implementation of the STP, Units 1 and 2 site into the HEC-HMS hydrologic model was consistent with the corresponding description in the FHRR. The precipitation forcing, initial and boundary conditions, and parameter values in the model were consistent with the corresponding descriptions provided in the FHRR. However, while running the HEC-HMS model using the input files provided by the licensee, the staff found that the HEC-HMS software produced a warning message that the time interval used for the simulation was too long. The staff reduced the computational time interval to a more appropriate value consistent with HEC-HMS recommendation. The staff found that this change produced peak discharges as much as 28 percent greater than those produced using the licensee's HEC-HMS configuration. Consequently, the staff determined that increased peak flows should be investigated in the subsequent analysis of the LIP using the HEC-River Analysis System (HEC-RAS) (USACE, 2010b) to determine peak water surface elevations.

The licensee stated that the current NRC guidance (i.e., NUREG/CR-7046 (NRC, 2011e), RG 1.59 (NRC, 1977), and ANSI/ANS-2.80-1992 (ANS, 1992)) was the basis for its approach and methodology for reevaluating the LIP flooding hazard (STPNOC, 2013). However, during the review, the staff found that the licensee did not describe how it applied the hierarchical hazard analysis (HHA) approach described in NUREG/CR-7046. Because the licensee did not present details of the HHA process that resulted in the LIP flood model, the staff evaluated alternative conceptual models to understand the sensitivity of the flood water surface elevations during the LIP runoff event.

The licensee stated that 90 percent of the ground topography measurements associated with the aerial survey had an elevation uncertainty of 0.5 feet (ft)(0.15 m) and the rest 1.0 ft (0.30 m) (STPNOC, 2014b). Uncertainty in ground elevations could lead to uncertainties in estimation of slopes, times of concentration, shape of the hydrographs, and peak discharges. Because an increase in slope can result in shorter times of concentration and increased peak discharges, the staff evaluated the sensitivity of HEC-HMS predictions using estimated potential increases in slopes of all 14 subbasins. The staff estimated that the slope of the longest flow path, the STP drain, in the site contributing drainage area could change, with the elevation at the upstream end of the flow path being 0.5 ft (0.15 m) higher and the downstream end of the flow path being 0.5 ft (0.15 m) lower. With this estimated increase in slope, the staff found that times of concentration presented in the FHRR would decrease by 6 percent to 24 percent. Changes in the times of concentration for all subbasins were less than 7 percent except for one, which was 24 percent. The sub-basin with the 24 percent change to its time of concentration covers about 1 percent of the total area of the contributing drainage. Because the changes in times of concentration are relatively small and the one sub-basin that has an appreciable change to its time of concentration covers only a small fraction of the total area, the staff determined that the elevation uncertainly would not introduce appreciable changes in HEC-HMS simulated peak discharges.

The licensee described a process for combining up to three types of flow—sheet flow, shallow concentrated flow, and ditch flow—to estimate the time of concentration following NRCS's Technical Release (TR)-55 document (STPNOC, 2014b; NRCS, 1986). The licensee used topographic data to estimate slopes and flow lengths and site conditions to specify roughness parameters. Empirical equations from TR-55 were used to estimate the times of concentrations for the subbasins. The licensee conservatively reduced the times of concentration by 25 percent to account for nonlinear effects of the extreme rainfall event. The licensee further selected HEC-HMS model subbasin lag times as 60 percent of the already reduced times of concentration.

Based on the review summarized above, the staff determined that by using a more appropriate HEC-HMS computation time interval, the peak discharges obtained were up to 28 percent greater, and therefore were used by the staff for estimating corresponding water surface elevations.

2.2.3 Water Level Determination

The licensee stated that the methods it used in the flood reevaluation were the same as those used in the STP, Units 3 and 4 COLA. These methods are consistent with present-day regulatory guidance and NUREG/CR-7046 (NRC, 2011e). The staff's review of the STP, Units 3 and 4 COLA was based on guidance in Section 2.4 of the Standard Review Plan (SRP) (NRC, 2007). The staff guidance for flood estimation in the SRP is based on the methods described in Regulatory Guide 1.59 (NRC, 1977) supplemented by current engineering practice, which are described in NUREG/CR-7046.

As mentioned previously, water-surface elevations at the STP, Units 1 and 2 site were estimated using PMP peak discharges simulated by HEC-HMS. The licensee used HEC-RAS to estimate water-surface elevations at the STP, Units 1 and 2 site. HEC-RAS cross sections developed by the licensee incorporate topographic information collected to support the STP, Units 3 and 4 COLA (STPNOC, 2013). The licensee's reaches and cross sections are shown in Figure 3.2-3 and Figure 3.2-4. The licensee stated that buildings and other structures in the flow path were accounted for by incorporation of obstructions in the HEC-RAS model. The licensee assumed that all culverts and stream channels were blocked during the PMP event.

The primary HEC-RAS reach, STP drain, flows east from where the Little Robbin Slough meets Farm-to-Market Road 521 to Kelly Lake (STPNOC, 2013). The HEC-RAS model was configured to allow for lateral weir overflows to Little Robbins Slough as a secondary HEC-RAS flow path southward from West Access Road. The HEC-RAS model contains five internal weirs, which are used to represent four roads and a concrete vehicle barrier near the power block area (see Figure 3.2-3). The division of flow at the model junctions was computed by HEC-RAS using a split-flow option based on maintaining an energy balance as flows are divided into multiple downstream flow paths. Normal flow boundary conditions are used in the downstream end of both modeled reaches. The licensee assigned Manning's roughness coefficient values of 0.040 for natural channels and 0.033 for excavated channels. The licensee assigned Manning's roughness coefficient values of 0.160, 0.050, 0.016, and 0.025 for dense brush, high grass, concrete and asphalt, and gravel surfaces, respectively. The licensee stated that these values are consistent with values of Manning's roughness coefficient reported in literature. The licensee treated all areas as being nearly impervious by specifying a runoff curve number of 98.

For select cross sections that intersect the STP, Units 1 and 2 power block area, Table 3.2-5 provides the licensee's local intense precipitation peak water-surface elevations computed using HEC-RAS. These range from 32.9 ft (10.0 m) NGVD29 to 33.0 ft (10.1 m) NGVD29.
Given the significant role that the HEC-RAS model performed in the licensee's analysis of the PMF from LIP and the need to review the formulation of the model's complex spatially and

temporally distributed input, the staff reviewed the HEC-RAS model input files provided by the licensee (STPNOC, 2014a). Based on the review, the staff determined that the licensee's implementation of the STP, Units 1 and 2 site into the HEC-RAS hydraulic model was consistent with the corresponding description in the FHRR. The licensee used two alternative conceptual models that were implemented in the HEC-RAS model. The two conceptual models differ in the way the flow is routed around the ECP. The licensee conducted a steady-state analysis using peak discharges estimated from the HEC-HMS simulation for each subbasin. Regardless of the time the peak discharges occurred in the respective subbasins, the licensee's HEC-RAS model used them as steady discharges in the corresponding stream reaches. Both of these assumptions (using peak discharges regardless of when they occur in different subbasins and performing a steady-state simulation) are conservative for estimating water surface elevations within the STP, Units 1 and 2 site. The staff reviewed the boundary conditions used by the licensee and found them appropriate.

As stated in Section 2.2.2, the staff determined that the peak discharges estimated using the HEC-HMS simulation were lower because they were based on a computational time interval that was larger than HEC-HMS guidance. The staff found that selecting a more appropriate computation time interval in the HEC-HMS model resulted in up to 28 percent larger peak flow estimates. Therefore, the staff increased the licensee-specified peak discharges from each subbasin by 30 percent and performed a HEC-HMS simulation. The staff found that the maximum change in peak water surface elevation resulting from this change was 1.6 ft (0.49 m), which occurred at the downstream end of the main drainage ditch near Kelly Lake. Closer to STP, Units 1 and 2, the peak water surface elevation increased from 33.5 ft to 33.9 ft (10.21 m to 10.33 m). The licensee stated in the FHRR that the current design-basis flood elevations in the power block area vary from 44.5 ft to 50.8 ft (13.56 m to 15.48 m) and that, for the ECWIS, the elevation is 40.8 ft (12.44 m). Therefore, the staff determined that no conclusions made in the FHRR would be changed based on the use of a more appropriate HEC-HMS computational time interval.

The staff also performed a sensitivity analysis by altering the values of Manning's n in the licensee's HEC-RAS simulations. To examine the sensitivity of predicted water surface elevations near the power block area, the staff increased the values of Manning's n for reaches upstream of the power block area and reduced those for reaches downstream of the power block area. The staff used the minimum and maximum of the range of Manning's n values suggested in literature for each surface type while reducing or increasing these values, respectively. Using the altered values of Manning's n, the staff found that the water surface elevation could change from 0.3 ft to 1 ft (0.09 m to 0.30 m).

Given the significant role that elevation data have in defining slope and flow paths, the staff requested the licensee provide a description of the methods used to incorporate elevation measurements and associated errors into the HEC-RAS and HEC-HMS analyses. The licensee stated that more than 90 percent of STP ground topography measurements associated with an aerial survey were accurate to ±0.5 ft (0.15 m) and the remaining measurements were accurate to ±1.0 ft (0.30 m). The licensee stated that the maximum LIP flood water surface elevation was determined to be more than 10 ft (3.05 m) lower than the design-basis flood water surface elevation. As stated in Section 3.2.2, the staff determined that the change in slope within the contributing drainage area is not significant to affect peak discharge.

2.2.4 Conclusions

The staff confirmed the licensee's conclusion that the reevaluated flood hazard for LIP and associated site drainage is bounded by the current design-basis flood hazard.

2.3 Streams and Rivers

The licensee reported in its FHRR that the reevaluated probable maximum flood elevation, including associated effects, for site flooding from stream and rivers is based on a stillwater surface elevation of 26.3 ft (8.00 m) NGVD29 (STPNOC, 2013). This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis probable maximum flood elevation for site flooding from streams and rivers is a stillwater-surface elevation of 29 ft (8.8 m) NGVD29.

The staff describes its evaluation of site flooding from streams and rivers, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

2.3.1 Probable Maximum Precipitation

The STP, Units 1 and 2 site location in the Lower Colorado River Basin (LCRB) is shown in Figure 3.3-1 (FHRR Figure 2.2-1a); the locations of additional hydrologic features and dams are shown in Figure 3.3-2 (FHRR Figure 2.2-1b). The licensee used the HEC-HMS software to develop estimates of peak discharge for the 80 subbasins that comprise the LCRB (STPNOC, 2013). The licensee adopted a subbasin delineation that was used for a previous flood study conducted by Halff Associates, Inc. (Halff, 2002). The subbasins are shown in Figure 3.3-3 (FHRR Figure 2.2-2a).

The licensee stated that HMRs 51 (Schreiner and Riedel, 1978) and 52 (Hansen, Schreiner, and Miller, 1982) were used to estimate PMP depths (STPNOC, 2013). The licensee examined meteorological data recorded since the HMRs were published and found no information to suggest the PMP has been exceeded. The licensee stated that the historical maximum 24-h point rainfall depth was 43 in (109.2 cm) and was recorded at Alvin, Texas on July 25 through 26, 1979 (STPNOC, 2013). As described in Section 3.2, this value was apparently from a post-storm survey; the staff found that the NOAA State Climate Extremes Committee website reports a maximum measured point 24-h rainfall of 42 in (106.7 cm) at Alvin, Texas on July 25 through 26, 1979 (NOAA, 2013). This historical maximum 24-h point rainfall depth is less than the 10-mi^2 (26-km^2), 24-h PMP depth of 47.1 in (120 cm) that was developed for the STP, Units 1 and 2 analyses.

The licensee stated that the approach and methodology used for the estimation of the PMF in the STP, Units 3 and 4 COLA was adopted for the reevaluation. The staff has reviewed the STP, Units 3 and 4 COLA and accepted the PMP used for the drainage area (NRC, 2012e). Therefore, the licensee used a reasonable approach and methodology for estimating the PMF for the reevaluation.

2.3.2 Runoff and Stream Course Models

The licensee used the HEC-HMS software (USACE, 2010a) to evaluate the watershed runoff from the PMP (STPNOC, 2013). The HEC-HMS model is adapted from the HEC-HMS model developed by Halff (hereafter referred to as the Halff HEC-HMS model) for the lower part of the LCRB (Halff, 2002).

The Halff HEC-HMS model incorporates no initial loss and a small uniform precipitation loss. The licensee stated that initial precipitation loss was set to zero in the HEC-HMS model for each of the 80 subbasins and a uniform loss rate of 0.05 in/h (0.1 cm/h) was used for the PMF analysis (STPNOC, 2013). The licensee stated that the value was a conservative estimate of loss rate and consistent with guidance provided by the Federal Energy Regulatory Commission (FERC, 2001) and NUREG/CR-7046.

The Halff HEC-HMS model uses the HEC-HMS Snyder unit hydrograph option for development of subbasin drainage hydrographs. The licensee decreased the basin lag time parameter of the Snyder unit hydrographs by 25 percent to account for the nonlinearities associated with the extreme conditions during a PMF event. The licensee also adjusted 3 of the 58 storage-outflow channel rating curves incorporated into the HEC-HMS model to accommodate the extreme conditions that the Halff HEC-HMS model was not originally designed to handle.

The licensee used HEC-RAS software to estimate the PMF water-surface elevations near the STP, Units 1 and 2 site. The HEC-RAS model covers the area from Bay City to Matagorda Bay (STPNOC, 2013). The licensee adopted the Halff HEC-RAS model (Halff, 2002). The licensee stated in the FHRR that channel cross sections were developed based on topographic information supplied by the USACE and based on aerial orthophotographs, 2-ft (0.6-m) interval contour maps, and National Elevation Dataset Digital Elevation Model data.

The licensee stated (STPNOC, 2013, Section 2.2.5) that channel and floodplain roughness coefficients used in the HEC-RAS model were set according to U.S. Geological Survey National Land Cover Dataset coverages and adjusted based on aerial photographs used in the Halff HEC-RAS model (Halff, 2002). The licensee stated that roughness coefficient values were also adjusted using model calibration. Halff-calibrated Manning's roughness coefficient values were 0.035 for the river channel, 0.045 to 0.05 for the overbank, and 0.085 to 0.095 for the floodplain (Halff, 2002). The licensee stated that the Halff-calibrated roughness coefficient values were increased by 20 percent in the reevaluation to account for extreme PMF conditions when discharge in the Lower Colorado River would occur over floodplain areas that were higher and rougher than those that the Halff HEC-RAS model was expected to simulate.

The licensee stated that the HEC-RAS model domain did not extend downstream into tidally controlled areas (STPNOC, 2013). The licensee adopted a normal depth downstream boundary condition using an estimated channel slope of 0.0001 and the peak PMF discharge described below. The licensee prescribed a downstream water-surface elevation of 17.5 ft (5.33 m) NAVD88 or 17.3 ft (5.27 m) NGVD29 for a steady-state HEC-RAS simulation.

2.3.3 Probable Maximum Flood Flow

The licensee's reevaluation of the PMF in the LCRB was based on three scenarios:

- Scenario 1 considered a PMF in the 3,555-mi^2 (9,207-km^2) drainage area between Mansfield Dam and Bay City combined with a 90,000 ft^3/s (2,500 m^3/s) discharge from the Mansfield Dam and a baseflow of 5,200 ft^3/s (150 m^3/s) at Bay City (STPNOC, 2013). The licensee used an antecedent storm, equivalent to 40 percent of the PMP storm, three days prior to PMP event. The licensee estimated that Scenario 1 peak PMF discharge at Bay City would be 1,397,432 ft^3/s (40,000 m^3/s).
- Scenario 2 considered a PMF inflow to Mansfield Dam resulting from a PMP in the upstream drainage area, from Lake O.H. Ivie to Mansfield Dam, routed through Lake Travis and combined with the flood discharge from a storm equal to 40 percent of the PMP occurring three days after the upstream PMP, in the 3,555-mi^2 (9,207-km^2) drainage area between Mansfield Dam and Bay City and a baseflow of 5,200 ft^3/s (150 m^3/s) at Bay City (STPNOC, 2013). The licensee estimated that Scenario 2 peak PMF discharge at Bay City would be 1,252,615 ft^3/s (35,470 m^3/s).
- Scenario 3 considered the PMF in the 18,197-mi^2 (47,130-km^2) drainage area between Lake O.H. Ivie and Bay City combined with the flood generated from a standard project storm occurring three days prior to the PMP event over the whole area and a baseflow of 5,200 ft^3/s (150 m^3/s) at Bay City (STPNOC, 2013). The licensee considered no Lake Travis storage effects in this scenario; flows are considered unregulated. The licensee estimated that Scenario 3 peak PMF discharge at Bay City would be 994,060 ft^3/s (28,000 m^3/s).

The licensee determined that Scenario 1 was the most critical scenario for STP, Units 1 and 2 producing a peak PMF discharge of 1,397,432 ft^3/s (40,000 m^3/s) at Bay City. Scenario 1 was used by the licensee to determine the maximum stillwater-surface elevation at the STP, Units 1 and 2 site (STPNOC, 2013).

2.3.4 Water Level Determinations

The licensee used the HEC-RAS model to determine the maximum stillwater-surface elevation at STP, Units 1 and 2 site for two cases (STPNOC, 2013).

In the first case, the licensee increased the values of the roughness coefficient by 20 percent from the values used in the Halff study (Halff, 2002). The licensee's reevaluated peak stillwater-surface elevation near STP, Units 1 and 2 was determined to be 26.1 ft (7.96 m) NAVD88, or 26.3 ft (8.02 m) NGVD29.

In the second case, the roughness coefficients from the Halff study were used, and the stillwater-surface elevations were 1.3 ft (0.40 m) lower. The licensee stated that Revision 15 of the STP, Units 1 and 2 UFSAR (STPNOC, 2010) indicates a stillwater-surface elevation for PMF in streams and rivers 2.7 ft (0.82 m) higher than the reevaluated stillwater-surface elevation for the first case.

2.3.5 Coincident Wind-Wave Activity

The licensee did not estimate coincident wind-wave activity associated with the PMF in the Lower Colorado River (STPNOC, 2013). The licensee stated that the stillwater-surface elevation resulting from the upstream dam failures exceeded that resulting from the PMF.

Therefore, the licensee estimated the coincident wind-wave activity in the Lower Colorado River only for the upstream dam failures evaluation, which is described in Section 3.4.

2.3.6 Conclusions

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from streams and rivers is bounded by the current design-basis flood hazard.

2.4 Failure of Dams and Onsite Water Control/Storage Structures

The licensee reported in the FHRR that the reevaluated PMF elevation, including associated effects, for site flooding due to failure of dams and onsite water control or storage structures is 42.5 ft (13.0 m) NGVD29 at the STP, Units 1 and 2 power block and 35.4 ft (10.8 m) NGVD29 at the ECWIS (STPNOC, 2013). This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis PMF elevation for site flooding due to failure of dams and onsite water control or storage structures is 44.5 to 50.8 ft (13.6 to 15.5 m) NGVD29 at the STP, Units 1 and 2 power block and 40.8 ft (12.4 m) NGVD29 at the ECWIS.

In addition to the FHRR, the staff reviewed calculation packages and supplemental information via an online electronic reading room provided by the licensee.

To supplement the FHRR, the staff issued RAIs to the licensee. The licensee's responses (STPNOC, 2014a, b) provided details on the handling of ineffective flow areas and levees in models; details of inter-basin flows in models; electronic versions of model input files; and additional details on modeling of breach outflows.

The staff describes its evaluation of site flooding from failure of dams and onsite water control or storage structures, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

2.4.1 Upstream Dam Failures

The licensee stated that flooding at the STP site from failures of upstream dams on the Colorado River were analyzed previously for the STP, Units 1 and 2 UFSAR (STPNOC, 2010) and for the STP, Units 3 and 4 FSAR (STPNOC, 2012a). The licensee's flooding reevaluation described in the FHRR adopted the approach in the STP Units 3 and 4 FSAR (STPNOC, 2013).

2.4.1.1 Dam Failure Scenarios

The licensee considered two dam failure scenarios. For both scenarios, the licensee neglected upstream and off-channel dams with small potential flood volumes.

- Scenario 1 is based on simultaneous seismically induced failure of all upstream dams combined with (1) wave activity from a two-year wind event and (2) either a 500-year flood or 50 percent PMF, whichever is less, as recommended by ANSI/ANS-2.8-1992 (ANS, 1992).

- Scenario 2 is based on domino-type dam failures combined with (1) wave activity from a two-year wind event and (2) either a 500-year flood or 50 percent PMF, whichever is less, as recommended by ANSI/ANS-2.8-1992 (ANS 1992). In this scenario, all dams upstream of Buchanan Dam with storage volumes greater than 5,000 acre-ft (6,200,000 m^3) would fail such that their combined flood discharge would arrive at Buchanan Dam simultaneously and trigger the failure of Buchanan Dam. The failure of Buchanan Dam would cause a flood that would result in the overtopping of Mansfield Dam and its subsequent failure.

The licensee determined that Scenario 2 generated the bounding flood conditions at the STP, Units 1 and 2 site.

2.4.1.2 Conceptual Unsteady Flow Model

The licensee used the HEC-RAS software to simulate the dam breach flood routing in the Lower Colorado River system (STPNOC, 2013). The licensee determined that a 500,000 ft^3/s (14,000 m^3/s) constant flow was slightly greater than the peak Standard Project Flood inflow into the Buchanan Dam and the 500-year peak flood inflow into Mansfield Dam. Therefore, the licensee conservatively used the constant 500,000 ft^3/s (14,000 m^3/s) in combination with the postulated dam failure scenario.

2.4.1.3 Dam Data and Breach Sections

To characterize the breached sections of Buchanan and Mansfield Dams, the licensee used guidance from FERC (1991). The Buchanan Dam breach width was evaluated to be 1,470 ft (448 m) extending vertically from the top to the bottom of the dam, assuming 0.1 h for the breach to develop completely (STPNOC, 2013). The Mansfield Dam breach width was evaluated to be 1,360 ft (415 m) extending vertically from the top to the bottom of the dam, also assuming 0.1 h for the breach to develop completely.

2.4.1.4 Channel Geometry

The licensee incorporated site topography into the HEC-RAS model through characterization of the HEC-RAS channel geometry based on that developed in previous flood studies (Halff, 2002), which used elevation data referenced in NAVD88 (STPNOC, 2013). The prior study used a 474-mi (763-km) river reach from Texas Highway 190 upstream of the Buchanan Dam to about 1 mi (1.6 km) upstream of the Gulf Intracoastal Waterway.

The HEC-RAS model used to assess flooding from upstream dam breaches for the STP, Units 1 and 2 site covered the lower 414 mi (666 km) covered by the prior Halff study. The STP, Units 1 and 2 HEC-RAS model has an upstream boundary at Lake Buchanan. The licensee stated that all constrictions associated with bridge crossings were removed in the STP, Units 1 and 2 HEC-RAS model. The licensee also excluded some ineffective flow areas and levees where appropriate. The cross-section locations that were used in the STP, Units 1 and 2 analysis are shown in Figure 3.4-1 (FHRR Figure 2.3-4).

The licensee stated that the ineffective flow areas, including those that were behind levees, were removed from the Halff model because the discharge during the flood from the postulated breaches of upstream dams was expected to be greater than the standard project flood for which the Halff model was set up (STPNOC, 2014b). Because of the greater discharge, flood flow would occur in these previously ineffective areas.

To accommodate the storage volume necessary for the analysis of the postulated dam failure scenario, the licensee added cross sections to the Halff HEC-RAS model upstream of Buchanan Dam (STPNOC, 2013). The cross-sectional geometry was also modified to allow for intra-basin flows during extreme flood conditions; these intra-basin flows can occur over poorly defined drainage divides because of low relief in the LCRB. However, the NRC staff noted that allowing the intra-basin flows would reduce the flood discharge at the STP, Units 1 and 2 site and therefore the staff requested the licensee to provide a justification for allowing the intra-basin flows (NRC, 2014).

In response to the RAI, the licensee stated that the local drainage divide elevations near Garwood, Texas, during an initial HEC-RAS simulation were lower than the water surface elevations during the flood from the postulated breaches of upstream dams. Therefore, the licensee extended the cross sections near this location on both sides of the Lower Colorado River to represent more realistic flow conditions in the HEC-RAS model (STPNOC, 2014b).

After reviewing the licensee's model input files (STPNOC, 2014b), the staff finds that the licensee's implementation of the model in HEC-RAS is consistent with the description of the conceptual model described in the FHRR. The staff also reviewed the licensee's justifications for removing ineffective areas and accounting for inter-basin spillage, and determined that the justifications are reasonable and acceptable.

2.4.1.5 Channel Roughness

In the Halff HEC-RAS model, the Manning's roughness coefficient values were calibrated using observed storms (Halff, 2002). In the reevaluation described in the FHRR, the licensee increased the Manning's roughness coefficient values by a factor of two over those used in the Halff HEC-RAS model for the river reaches 4 mi (6 km) downstream of Mansfield and Buchanan Dams (STPNOC, 2013). This adjustment was made to account for increased roughness associated with entrained debris in extreme discharges resulting from dam breaches and for the likely overflow of the main channel with flow occurring in the floodplain, which has considerably greater roughness. Throughout the rest of the HEC-RAS model, roughness was increased by 20 percent over that used in the Halff study.

During the review of the licensee's HEC-RAS input files, the staff found that the values of Manning's n at a few locations were unusually high for natural channels. The staff performed a sensitivity simulation by setting these unrealistic values to more reasonable values, and found that the floodwater surface elevation adjacent to the STP site increased only slightly (0.7 ft [0.2 m]). Therefore, the staff concluded that the few unrealistically specified Manning's n values would not alter the licensee's conclusions reached in the FHRR.

2.4.1.6 Stillwater-Surface Elevations

The licensee stated that the estimated dam-failure stillwater-surface elevation would be 28.6 ft (8.71 m) NGVD29, or 28.4 ft (8.66 m) NAVD88, at the STP, Units 1 and 2 site. The licensee also stated that the time required for the peak discharge to occur at the STP, Units 1 and 2 site after a Mansfield Dam failure was estimated to be 65 h. For a sensitivity case where the licensee used baseline Manning's roughness coefficient values from the Halff HEC-RAS model (i.e. not increased by 20 percent) in all reaches except the two 4-mi (6-km) reaches immediately below the Buchanan and Mansfield Dams, the corresponding time for the peak discharge to occur at the STP, Units 1 and 2 site was 58 h.

The licensee noted that the two most critical upstream dam failure scenarios used in the STP, Units 1 and 2 UFSAR (STPNOC, 2010) resulted in estimated stillwater-surface elevations of 32 and 34.1 ft (9.8 and 10.4 m) NGVD29. Therefore, the STP, Units 1 and 2 UFSAR stillwater-surface elevations for upstream dam failures are 3.4 and 5.5 ft (1.0 and 1.7 m) higher than the reevaluated stillwater-surface elevation described in the FHRR.

To investigate the sensitivity of the floodwater surface elevation in the Lower Colorado River at the STP site to the timing of peak discharge from tributaries, the staff performed a simulation by specifying the tributary inflows as steady discharges equal to the corresponding tributary's peak discharge. The staff found that the resulting water surface elevation in the Lower Colorado River adjacent to the STP site was higher than the licensee's simulation. However, the staff's sensitivity case water surface elevations remained significantly lower than the current design-basis flood elevations.

2.4.1.7 *Wind Setup*

The licensee used a 2-year design windspeed of 50 mi/h (80 km/h) coincident with the upstream dam failures to estimate wind setup. Consistent with the recommendations for wind wave estimation methods in NUREG/CR-7046, the licensee used the USACE Coastal Engineering Manual (Lockhart and Morang, 2002; Pope and Lockhart, 2003; Demirbilek and Vincent, 2008; Resio et al., 2008; Scheffner, 2008; Sorensen and Thompson, 2008; and Burcharth and Hughes, 2011) and other water-wave references to estimate wind setup over two fetch lengths, one 15.5 mi (24.9 km) and the other 17.6 mi (28.3 km). The licensee estimated a maximum wind setup of 3.0 ft (0.91 m). The licensee stated that the STP, Units 1 and 2 UFSAR reported an estimated wind setup of 1.6 ft (0.49 m). The licensee determined that the maximum water-surface elevation expected at the STP, Units 1 and 2 site considering wind setup would be 31.6 ft (9.63 m) NGVD29.

2.4.1.8 *Wave Run-up*

The licensee stated that the STP, Units 1 and 2 power block grade is 28.0 ft (8.53 m) NGVD29 (STPNOC, 2013). Therefore, the licensee determined that the maximum water depth in the STP, Units 1 and 2 power block area due to upstream dam failures including coincident wind setup would be 3.6 ft (1.1 m). The licensee stated that, at this depth, wind waves would break. The licensee assumed the wind waves would approach the site such that maximum wave run-up would occur. Under these assumptions, the licensee determined that maximum wave run-up would be 4.7 ft (1.4 m) at site SSCs. Therefore, the licensee stated that the maximum water-surface elevation at the SSCs from upstream dam failure-induced flooding, including coincident wind setup and wave run-up, would be 36.2 ft (11.0 m) NGVD29.[3] The licensee stated that the corresponding maximum water-surface elevation, including wind setup and wave run-up, in the STP, Units 1 and 2 UFSAR is 43.7 ft (13.3 m) NGVD29 which is more than 7 ft (2.1 m) higher than the reevaluated estimate.

The licensee stated that the wind setup for the ECP embankment would be 3 ft (0.9 m), based on the embankment toe elevation of 27 ft (8.2 m) NGVD29, crest width of 6 ft (1.8 m), and outer slope of 3(H):1(V) (STPNOC, 2013). The licensee estimated a maximum wave run-up of 11.8 ft (3.60 m). Therefore, the licensee concluded that the waves would overtop the ECP embankment crest, which is at an elevation of 34 ft (10.4 m) NGVD29.

To estimate the wave run-up on the ECWIS, the licensee estimated the height of the transmitted wave into the ECP as the waves would overtop the ECP embankment and would fill up the ECP to its crest elevation (STPNOC, 2013). The maximum wave height of the transmitted wave was estimated to be 1.2 ft (0.37 m), which would result in a wave run-up of 1.8 ft (0.55 m) on the ECWIS, with a corresponding maximum water-surface elevation of 35.8 ft (10.9 m) NGVD29. The licensee also stated that the corresponding STP, Units 1 and 2 UFSAR estimate for the maximum water-surface elevation at the ECWIS is 39.3 ft (12.0 m) NGVD29, which is 3.5 ft (1.1 m) higher than the reevaluated estimate.

The staff determined that the reevaluated water-surface elevations at the safety-related SSCs of STP, Units 1 and 2 are lower mainly because of the lower water-surface elevations predicted by the licensee's updated HEC-RAS model. The staff's sensitivity analyses also demonstrated that the flood water surface elevation in the Lower Colorado River adjacent to the STP site would not change appreciably.

2.4.2 Main Cooling Reservoir Embankment Failure

The licensee examined flood effects of a breach of the embankment surrounding the MCR in three contexts:

- Updated safety review of operating STP, Units 1 and 2, using three different models
- COLA safety review of proposed future STP, Units 3 and 4, using two different models
- 2012 studies for better understanding of flood effects, including debris, on operating STP, Units 1 and 2

2.4.2.1 Updated Safety Review of Operating STP, Units 1 and 2 in the UFSAR

The licensee previously examined an MCR embankment failure in the UFSAR for STP, Units 1 and 2; this is the flooding mechanism that controls the DBF (STPNOC, 2013). The locations of STP, Units 1 and 2 relative to the MCR are shown in Figure 3.4-2 (FHRR Figure 2.3-21).

[3] Adding the values given from this paragraph, 28.0 ft elevation + 3.6 ft setup + 4.7 ft wave run-up gives a total of 36.3 ft rather than 36.2 ft. This difference appears to result from rounding, and staff does not consider it significant.

The STP, Units 1 and 2 UFSAR evaluation was based on the use of three models (STPNOC, 2013):

- The Danish Hydraulic Institute System 21 software was used for the first model. The critical embankment breach width that would result in the maximum flood water-surface elevation in the STP, Units 1 and 2 power block area and at the ECP was determined using unsteady flow simulations.
- The Danish Hydraulic Institute System 21 software was also used for the second model. The licensee performed quasi-steady-state simulations over a finer-resolution computational grid covering the STP, Units 1 and 2 power block and the ECP to estimate maximum floodwater-surface elevations.
- The National Weather Service DAMBRK software was used in the third model. It was used to estimate the effects of the MCR embankment breach flood on the southern embankment of the ECP.

The licensee concluded from these analyses that the maximum flood water-surface elevations at the power block area and the ECWIS were 50.8 ft (15.5 m) NGVD29 and 40.8 ft (12.4 m) NGVD29, respectively, from an instantaneous and total removal of a 1,890-ft (576-m)-wide embankment section.

2.4.2.2 <u>COLA Safety Review of Proposed STP, Units 3 and 4 in the FSAR</u>

A breach of the MCR embankment was also evaluated by the licensee as part of the STP, Units 3 and 4 COLA FSAR (STPNOC, 2012a). The results of the NRC staff's review of the STP, Units 3 and 4 MCR embankment breach and the subsequent flooding were documented in the STP, Units 3 and 4 Safety Evaluation Report (NRC, 2012e).

STPNOC, in its COLA analysis of the MCR embankment breach, used two approaches to characterize the discharge hydrograph following the postulated breach initiated by a piping mechanism (STPNOC, 2012a).

In the first approach, MCR embankment breach parameters were estimated using empirical equations. STPNOC used these breach formation parameters to specify the dynamics of breach development in the National Weather Service (NWS) FLDWAV model (Fread and Lewis, 1998). The NWS FLDWAV model was used to estimate the breach outflow hydrograph (the FLDWAV hydrograph hereafter) for the STP, Units 3 and 4 COLA analysis.

In the second approach, STPNOC used the NWS BREACH model (Fread, 1991), which includes a process model of breach formation, to estimate breach development dynamics and the outflow hydrograph (the NWS BREACH hydrograph hereafter). STPNOC concluded that the FLDWAV hydrograph was more conservative than the NWS BREACH hydrograph because the former had a greater peak discharge and a shorter time-to-peak. Based on this finding, STPNOC used the FLDWAV hydrograph in its design-basis calculation.

The staff, in its STP, Units 3 and 4 COLA review, performed an independent MCR embankment breach analysis using the NWS BREACH model (NRC, 2012e). The staff concluded that the FLDWAV hydrograph was conservative because a reasonably conservative set of NWS BREACH parameters resulted in an outflow hydrograph with a smaller peak discharge and a longer time-to-peak (NRC, 2012e).

2.4.2.3 *<u>2012 Studies for Better Understanding of Flood Effects for STP, Units 1 and 2</u>*

The licensee refers to another MCR breach analysis as the "2012 MCR Breach Flooding Analysis for STP 1 and 2" (STPNOC, 2013). This analysis was based on two simulation models: (1) the NWS BREACH model (Fread, 1991) to establish breach characteristics and the outflow hydrograph and (2) the RMA2 model (Donnel et al., 2008) for estimating the effects of the breach outflow at the STP, Units 1 and 2 site.

<u>NWS BREACH Model.</u>
The licensee applied the NWS BREACH model (Fread, 1991) to the MCR embankment to develop breach characteristics. The licensee stated that the NWS BREACH model configuration was the same as that used for STP, Units 3 and 4 FSAR.

The licensee's MCR NWS BREACH analysis showed a peak outflow discharge of 83,200 ft^3/s (2,356 m^3/s) occurring about 6.5 h after breach initiation (STPNOC, 2013). At peak outflow discharge, the breach bottom width was estimated to be 361 ft (110 m); the breach expanded to is final value of 448 ft (137 m) 30 h after initiation. The licensee's outflow hydrograph is the same as reported for the applicant's use of the NWS BREACH model in the STP, Unit 3 and 4 FSAR (the NWS BREACH hydrograph). The licensee used the NWS BREACH hydrograph at each of three postulated alternative MCR embankment breach locations; the three breach locations were used to estimate the effects of the flood on STP Unit 1, STP Unit 2, and the ECP.

<u>RMA2 Model.</u>
The licensee described the use of RMA2 (Donnel et al., 2008), a two-dimensional numerical flow modeling software, to estimate the spread of the MCR embankment breach outflow (STPNOC, 2013). The licensee stated that the use of the RMA2 software is similar to that described for the STP, Units 3 and 4 FSAR. The licensee stated that the location of the MCR embankment breach and the use of the NWS BREACH hydrograph rather than the FLDWAV hydrograph are the notable differences between the recent reevaluation and the STP, Units 3 and 4 FSAR evaluation. In the reevaluation, the breach hydrograph was used at three locations selected to maximize peak water-surface elevations at STP, Unit 1, STP, Unit 2, and the Nuclear Support Center (NSC) Building. The third scenario, where the breach is directed at the NSC Building, is designed to conservatively estimate the peak water-surface elevations at the ECP, which is directly behind (north of) the NSC Building. These three postulated breach locations are shown in Figures 3.4-3, 3.4-4, and 3.4-5 (FHRR Figure 2.3-27a, b, and c).

The licensee reported the maximum water-surface elevation time-history for each of the three breach scenarios at ten selected locations. These ten locations are shown in Figure 3.4-6 (FHRR Figure 2.3-28). The licensee-determined maximum water-surface elevation for each scenario at each location is provided in Table 3.4-1 (FHRR Table 2.3-8).
The licensee-determined duration of the inundation to a depth of 0.25 ft (0.076 m) or greater for each scenario at each of the ten locations is shown in Table 3.4-2 (FHRR Table 2.3-11).

The licensee estimated that a maximum water-surface elevation of 42.5 ft (13.0 m) NGVD29 would occur at the southern face of STP, Unit 1. The licensee estimated a maximum water-surface elevation of 35.4 ft (10.8 m) NGVD29 at the ECWIS. Both of the maximum water-surface elevation estimates were associated with the MCR embankment breach location south of STP, Unit 1.

Given the importance of the characterization of the spreading of the MCR embankment breach outflow, the staff reviewed the RMA2 model scenarios provided by the licensee (STPNOC, 2014a). The licensee provided three RMA2 scenarios corresponding to the three postulated locations of the breach in the MCR embankment. All of these scenarios used the NWS BREACH hydrograph as the boundary condition for RMA2 simulations. The staff was able to reproduce the licensee's RMA2 runs.

The staff had stated in the previous STP, Units 3 and 4 COLA review that the NWS BREACH model used with site-specific data and reasonable and conservative parameters is an acceptable approach for determining the flood discharge following a postulated MCR embankment breach (NRC, 2012e). The staff's sensitivity analysis with the licensee's RMA2 model shows only a slight rise in water surface elevation (0.5 ft (0.2 m) for a 30 percent increased discharge), which is within the margin between the current design-basis (44.5 to 50.8 ft [13.6 to 15.5 m] NGVD29) and the reevaluated hazard (42.5 ft (13.0 m) NGVD29) in the power block area. For these reasons, the staff finds the licensee's FHRR analysis for MCR embankment breach outflow acceptable.

To summarize, the licensee compared the results from the recent reevaluation of the MCR embankment breach with those from previous evaluations. The reevaluated peak water-surface elevations are lower than the current DBF water-surface elevations. The licensee concluded that the reevaluated peak water-surface elevations are lower because the reevaluation is based on a more realistic, but still conservative, approach. The licensee stated that there are significant margins between the DBF water-surface elevations and those expected from the reevaluated MCR embankment breach.

2.4.3 Conclusions

The staff confirmed the licensee's conclusion that the reevaluated flood hazard for failure of dams and onsite water control or storage structures is bounded by the current design-basis flood hazard.

2.5 Storm Surge

The licensee reported in the FHRR that the reevaluated probable maximum flood elevation, including associated effects, for site flooding due to storm surge is 29.3 ft (8.93 m) NGVD29. This flood-causing mechanism is described in the licensee's current design-basis. The current design-basis probable maximum flood elevation for site flooding due to storm surge is 26.74 ft (8.15 m) NGVD29.

The staff describes its evaluation of site flooding from storm surge, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

2.5.1 Overview of Modeling Methods and Results

The licensee stated that the probable maximum storm surge (PMSS) flooding impact was evaluated for the STP, Units 1 and 2 site as described in the STP, Units 1 and 2 UFSAR, Section 2.4.5.2 (STPNOC, 2010) that used a combination of a one-dimensional bathystrophic model to simulate the storm surge offshore of the Colorado River mouth and a one-dimensional hydraulic model HEC-2 (USACE, 1980) to simulate the routing of the flood surge up the Colorado River.

The resulting PMSS stillwater level at the site was estimated at 26.78 ft (8.163 m) NGVD29, below the power block grade elevation of 28 ft (8.5 m) NGVD29.

For the storm surge reevaluation, wind-wave run-up (due to probable maximum hurricane (PMH)) were determined specifically for STP, Units 1 and 2 safety-related structures, based on the stillwater level predicted for the site using the ADCIRC (Advanced Circulation Model) storm surge model as documented in the STP, Units 3 and 4 COLA FSAR Section 2.4.5 (STPNOC, 2012a).

2.5.2 Parameters of Probable Maximum Meteorological Winds

The licensee establishes the probable maximum meteorological winds (PMMWs) using guidance found in NWS Technical Report 23 (NWS 23) (NOAA, 1979). A summary of the licensee's PMMW and PMH parameters is reported in STP, Units 3 and 4 FSAR Section 2.4S.5.1 and Table 2.4S.5-2 (STPNOC, 2012a).

The peripheral pressure is 30.12 in Hg (1020 millibars). The central pressure is 26.19 in Hg (886.9 millibars). Therefore, the PMH central pressure deficit (Δp) is estimated to be 3.93 in Hg (133 millibars). The radius of maximum winds had upper and lower limits of 5 and 21 nautical miles (9 and 39 km), respectively. The forward speed had upper and lower limits of 6 and 20 knots (11 and 20 km/h), respectively.

The staff used NWS 23 to independently estimate the PMMW for the STP site. The staff's estimates of the PMH parameters using guidance from NWS 23 are given in Table 3.5-1 below. The staff also used the NOAA hurricane database and other currently available information to assess the relative severity of the NWS 23 PMH. Based on PMH parameter values derived from NWS 23, the staff estimated that the maximum wind speed for a moving and a stationary hurricane at the STP site would be approximately 157.6 and 149.7 mi/h (253.5 and 240.8 km/h) (Category 5 and Category 4), respectively. The estimated stationary hurricane wind speed of 149.7 mi/h (240.8 km/h) is consistent with, but slightly lower than, the applicant's estimated range of 152 to 160 mi/h (245 to 257 km/h) (Category 5) in STP, Units 3 and 4 FSAR Table 2.4S.5-3 (STPNOC, 2012a).

2.5.3 Storm Surge Analysis

2.5.3.1 Storm Surge Model System

The numerical simulation model "Advanced Circulation (ADCIRC)", which is a hydrodynamic circulation code that simulates water level and current over an unstructured gridded domain, was used by the licensee to simulate the PMSS elevation at the STP site in support of the combined license application (COLA) for STP, Units 3 and 4, FSAR Section 2.4S.5 (STPNOC, 2012a). ADCIRC is also linked to a computer program called SWAN (Simulating Waves Nearshore) (TUDelft, n.d.) that calculates the wave-induced setup in addition to the wind-induced setup calculated by ADCIRC. The STWAVE (STeady State spectral WAVE) model was used to simulate nearshore wave transformation and generation (Smith and Sherlock, 2007; Smith, Sherlock, and Resio, 2001).

The staff's independent review finds that the USACE ADCIRC model has a long history of development, verification, and validation. The staff therefore finds that ADCIRC is an appropriate model for simulating storm surges from hurricane events.

2.5.3.2 Antecedent Water Level

The licensee stated that the antecedent water level, as defined in RG 1.59 (NRC, 1977), was estimated separately and used to establish the model initial water level. The PMH parameters (Δp, radius of maximum wind, forward speed, track direction), as described previously, were used to define the physical attributes of the PMH in the model.

For the STP site, the licensee estimated the 10 percent exceedance high spring tide from the tidal records at the NOAA Freeport, Texas tide gage station, the closest tidal station from STP, which has 21 years of data. The Freeport station is located approximately 45 mi (72 km) southeast of the site. The licensee's 10 percent exceedance high spring tide elevation analysis at this station was found to be 3.59 ft (1.09 m) NGVD29. Because the 10 percent exceedance high spring tide is estimated from tidal records, as recommended by RG 1.59 (NRC, 1977), no additional assessment for initial rise was performed by the licensee.

In addition to the 10 percent exceedance high spring tide and initial rise, the licensee considered the long-term trend observed in tide gage measurements to account for the expected sea level rise for the 100-year period. The licensee stated that the long-term sea level rise trend at Freeport, Texas, as estimated based on data from 1954 to 2006, is 1.43 ft (0.436 m) per century. Accordingly, a nominal long-term sea level adjustment was applied by the licensee to the 10 percent exceedance high tide level resulting in an antecedent water level of 5.1 ft NGVD29. This water level was converted by the licensee to approximately 4.9 ft (1.5 m) NAVD88 and was used by the licensee as the initial water level in the ADCIRC model simulations.

As in the case of the licensee's ADCIRC simulations, a sea level rise of 1.93 ft (0.588 m) NAVD88, an initial rise of 2.6 ft (0.79 m) NAVD88, and the 10 percent exceedance high tide of 2.2 ft (0.67 m) NAVD88 were added by the staff to the ADCIRC stillwater level calculations that included a wind wave and wave setup. This calculation was made using STWAVE/WAM, which combines the STWAVE model with WAM (Wave prediction Model; USACE, n.d.-b).

There was no adjustment equal to the difference between the 10 percent exceedance high tide level and mean tide level, thus adding additional conservatism.

2.5.3.3 Topographic and Bathymetric Data

The topography in Coastal Texas was mapped in the STP, Units 3 and 4 COLA ADCIRC hydrodynamic model by the licensee using topographic survey data obtained from a variety of data sources. Topographic data for the majority of the terrain in Texas were obtained from the Texas Water Development Board (TWDB, n.d.), Harris County Flood Control District (HCFCD, n.d.), and Texas Natural Resources Information System (TNRIS, n.d.). These data were available to the licensee in digital elevation model form at a 10-m by 10-m resolution, and some later became available at a 1-m by 1-m resolution.

Bathymetry in the portions of the western North Atlantic, Gulf of Mexico, and Caribbean Sea included in the models was drawn from sources including the raw bathymetric sounding database from the National Ocean Service (NOS), the Digital Nautical Charts (DNC) bathymetric database, and ETOPO5 data (NOAA, n.d.-b). The licensee stated that the bathymetry for inland waterways in Coastal Texas is provided by regional bathymetric surveys and dredging surveys from the USACE Southwest Galveston District (SWG), NOAA, Texas Water Development Board, or nautical charts.

2.5.3.4 Model Grid

The licensee stated that Version 13 of the Texas topographic grid, [herein referred to as TX2008 model grid], is an extension of the earlier EC2001 U.S. East Coast and Gulf of Mexico tide model grid and the TX2004 Coastal Texas storm surge model grid (Bailey et al., 2014).

The licensee stated that the TX2008 computational mesh grid contains more than 2.8 million nodes and nodal spacing varies significantly throughout the mesh. Grid resolution varies from approximately 12 to 15 mi (19 to 24 km) in the deep Atlantic Ocean to about 100 ft (30.5 m) in Texas.

Based on the above, the staff determines that the ADCIRC bathymetric and topographic data used by the licensee and contained in the TX2008 model are significantly more detailed than those used by the staff and USACE in an independent staff ADCIRC storm surge analysis for STP, Units 3 and 4. The primary difference the between the staff's and the licensee's model grid is the presence of the two topographic features (the City of Matagorda levee and the dredge pile) in the licensee's Texas Grid version 13 that are not represented in the staff grids. These two features are located southeast of the STP site and create a shadowing effect (i.e., lowering the storm surge water level) on the advancement of the licensee's ADCIRC storm surge from the Gulf toward the site.

2.5.3.5 Storm Surge Results

The licensee performed model simulations with numerous combinations of input PMH parameters to obtain the PMSS elevation. The effect of wind-wave run-up is superimposed on the PMSS elevation to obtain the maximum water level at the STP facilities.

The licensee's results from storm surge simulations using STP, Units 3 and 4 COLA FSAR Section 2.4S.6 (STPNOC, 2012a) ADCIRC model indicated that the maximum water surface elevation near the STP site would be produced by a large (in terms of radius to maximum winds), fast-moving (in terms of forward speed) storm that would produce prevailing winds blowing from the east or southeast toward the STP site. Therefore, the licensee stated that the PMSS of 29.3 ft (8.93 m) NGVD29 will occur as the result of a hurricane traveling towards northwest direction (i.e., an approach direction of 135 degrees clockwise from the north) passing within 24 mi (39 km) of the STP site. The licensee stated that before landfall the storm will have a constant forward speed of 23 mi/h (37 km/h), a central barometric pressure of 26.2 in Hg (887 millibars), and a maximum sustained wind speed (1-min average) of 184 mi/h [296 km/h]).

For STP, Units 3 and 4, the staff's stillwater PMSS for all storms never exceeds 30.9 ft (9.42 m) NGVD29.The total PMSS (including wave run-up) is 39.8 ft (12.13 m) NGVD29. Thus, the staff determines that the licensee's site-specific PMSS maximum water surface elevation of 29.3 ft (8.93 m) NGVD29 is reasonable and conservative.

For STP, Units 1 and 2, the safety related SSCs are the safety related structures and facilities in the power block, and the Essential Cooling Pond (ECP) and the Essential Cooling Water Intake Structure (ECWIS). The licensee appropriately applied the USACE Coastal Engineering Manual equations for wave run-up to the site specific structure characteristics for the STP, Unit 1 and 2 SSCs. For STP, Units 3 and 4, the staff's stillwater PMSS for all storms never exceeds 30.9 ft (9.42 m) NGVD29. The staff also concludes that the maximum PMSS water surface elevation at the STP, Units 3 and 4 site accounting for the wind setup and run-up would not exceed 30 ft (9.1 m) NGVD29.
The licensee used 29.3 ft (8.93 m) NGVD29 to estimate the significant, 2 percent and maximum wave run-up on the ECP embankment resulting in 32.5 ft, 33.7 ft, and 35.2 ft (9.91 m, 10.3 m, and 10.7 m) NGVD29, respectively. However, the staff concludes that the 0.7 to 1.6 ft (0.2 to 0.49 m) difference between the licensee and staff PMSS would have an insignificant impact on the aforementioned results. Note that stillwater PMSS in conjunction with wave effects is used to determined total wave run-up on site-specific structures.

The licensee stated that the maximum run-up level at the ECWIS as a result of PMH wind action, in combination with a 100-year 4-day precipitation, is predicted to be 40.1 ft (12.2 m) NGVD29 as part of seiche flooding evaluation described in FHRR Section 2.5.

2.5.4 Conclusion

The staff finds that the reevaluated hazard for flooding from storm surge is bounded by the current design-basis flood hazard.

2.6 Seiche

Seiche effects were evaluated separately for the ECP and the Main Cooling Reservoir. This flood-causing mechanism is not described in the licensee's current design-basis.

The staff describes its evaluation of site flooding from seiche, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

Essential Cooling Pond (ECP)
The licensee reported in the FHRR that the reevaluated probable maximum flood elevation, including associated effects, for site flooding due to seiche effects is 40.1 ft (12.2 m) at the ECWIS. The licensee stated that typical seismic wave periods are on the order of seconds, making it unlikely for seiching in the ECP to be amplified from seismic motions because of the significant difference between the forcing period and the ECP's natural period.

Main Cooling Reservoir (MCR)
The licensee stated that a quantitative analysis of the seiche motions and overtopping potentials in the MCR was not conducted in this evaluation because the MCR is not a safety related facility for STP, Units 1 and 2, and so the loss of cooling function as a result of seiche flooding would not affect the safety of the plant. The licensee concluded that, in the unlikely event of overtopping at the MCR embankment as a result of seiche motions, the flooding impacts would be bounded by those resulting from the failures of the MCR embankment. Failure of the MCR embankment is the design-basis flooding mechanism of STP, Units I and 2, as described in FHRR Subsection 2.3.2.
Conditions in other water bodies are not conducive to seiche motions.

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from seiche is bounded by the current design-basis flood hazard.

2.7 Tsunami

The licensee reported in the FHRR that the reevaluated probable maximum flood elevation, including associated effects, for site flooding due to tsunami is 11.5 ft (3.52 m) NGVD29. This flood-causing mechanism is described in the licensee's current design-basis, but was not evaluated.

The staff describes its evaluation of site flooding from tsunami, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

The licensee evaluated several different tsunami sources from the published scientific literature to establish the probable maximum tsunami (PMT) at the site. Approximate tsunami wave heights are indicated by Knight (2006) for four seismogenic sources located in the Caribbean and the Gulf of Mexico and by Mader (2001) for the 1755 Lisbon earthquake, which was located in the Atlantic Ocean. The wave height estimate from Trabant et al., (2001) for the East Breaks submarine landslide is considered highly unlikely by the licensee.

After reviewing published tsunami catalogs, databases, and historical accounts, the licensee identified the following three historical events as being relevant to the STP site:

- An October 11, 1918, seismogenic tsunami originating west of Puerto Rico.

- A May 2, 1922, seismogenic tsunami originating near the Virgin Islands.

- A March 27, 1964, Gulf of Alaska earthquake generating seismic seiche waves. This was not a tsunami event in the Gulf of Mexico.

The licensee examined published information to determine the source generator characteristics for several different types of potential tsunami sources: seismogenic, volcanogenic, and landslide generated. For seismogenic tsunamis, the licensee discussed the propagation characteristics into the Gulf of Mexico for earthquakes located in the Caribbean and the Atlantic Ocean (Knight, 2006). For volcanogenic tsunamis (catastrophic flank failures), the licensee cited recent studies to discount the La Palma, Canary Islands transoceanic tsunami scenario published by Ward and Day (2001). For landslide-generated tsunamis, the licensee discounted the East Breaks landslide tsunami scenario published by Trabant et al., (2001) as highly unlikely. The East Breaks landslide occurred at the edge of the continental shelf off Texas between 10,000 and 25,000 years ago.

To determine the maximum tsunami water levels, the licensee used an estimate of the tsunami in the Gulf of Mexico from a near-field submarine landslide near the East Break slump and then applied (1) a run-up amplification factor, (2) 10 percent exceedance of an astronomical high tide according to RG 1.59 (NRC, 1977), and (3) a sea level rise from global climate change in the next century. The licensee determined the maximum water level for the PMT to be 11.5 ft (3.51 m) NGVD29.

Therefore, the licensee concluded that the flood elevation at STP, Units 1 and 2 due to the postulated PMT event, will not be the controlling design-basis flood elevation for STP, Units 1 and 2 because it is below the plant grade, and because there will be no onsite effects from tsunami breaking waves or resonance or onsite tsunami waves on safety-related facilities.

The NRC staff reviewed the information in Section 3.7 of the STP FHRR that was derived from the STP, Units 1 and 2 COL FSAR (STPNOC, 2010). The staff's review confirms that the information provided by the licensee addressed the relevant information related to the PMT. Staff's independent confirmatory analysis of tsunami water levels at the STP site focused on distant earthquake tsunami sources and landslide sources local to the Gulf of Mexico.

Staff conducted an independent analysis of the 10 percent exceedance high tide, because this value is used in determining PMT. Staff's analysis is based on 16 years of data from the NOAA National Ocean Service Center for Operational Oceanographic Product Services (NOS-CO-OPS) at the Freeport tide gauge station (years 1992 through 2007), (NOAA, n.d.-a, accessed 2008). The 10 percent exceedance high tide is determined to be 1.48 ft (0.45 m) relative to the NGVD29 level for these years. This finding is consistent with the licensee's estimate of 1.51 ft (0.46 m) NGVD29, as is indicated in the STP Units 3 and 4 FSAR Section 2.4.6 (STPNOC, 2012a).

According to the staff analysis, the long-term sea level rise at the Freeport station is 0.171 ± 0.0441 in/yr (4.35 ± 1.12 mm/yr), according to the NOAA NOS-CO-OPS data. The estimate in the licensee's STP Units 3 and 4 FSAR, Section 2.4S5, is 0.231 ± 0.0291 in/yr (5.87 ± 0.74 mm/yr) (STPNOC, 2012a).

For STP, Units 1 and 2, the staff's estimated PMT water level is 16.53 ft (5.04 m) NGVD29. This is the sum of three components:

- 1.47 ft (0.45 m) NGVD29 (10 percent exceedance high tide)
- 13.12 ft (4 m) (maximum tsunami run-up)
- 1.94 ft (0.59 m) (sea level rise over one century)

For comparative purposes, the staff re-computed the offshore tsunami water levels for the northern Caribbean subduction zone and the northern South American convergent zone earthquake scenarios of ten Brink et al. (2008). These scenarios use the COMCOT model (Cornell, n.d.) that includes non-linear terms and a moving boundary condition at the shoreline and computes the model in spherical coordinates. Bottom friction is also included but is set at a low, conservative value ($f = 10^{-4}$) in this case.

Results of these simulations confirm that tsunami amplitudes from distant Caribbean earthquakes are less than 3.3 ft (1.0 m) near the STP site. Tsunami amplitudes from earthquakes along the Azores-Gibraltar oceanic convergence boundary are also likely to be less than 3.28 ft (1 m) in the Gulf of Mexico.

At the STP site, the bounding Main Cooling Reservoir breach water level is 40 ft (12.2 m) NGVD29, and the plant grade is 28 ft (8.53 m) NGVD29. The staff finds that the PMT water level is below these levels.

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from tsunami is bounded by the current design-basis flood hazard.

2.8 Ice-Induced Flooding

The licensee reported in its FHRR that the reevaluated probable maximum flood elevation, including associated effects, for ice-induced flooding does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The staff describes its evaluation of ice-induced flooding of the site, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

The licensee examined temperature data from the Lower Colorado River Authority (LCRA) web site (LCRA, n.d.) at three gaging stations (Bay City, Wharton, and Columbus) near STP. The available period of record was from 1982 through 2006. The minimum recorded water temperature was 41.2 °F on February 6, 1985.

The licensee searched the "Ice Jam Database" maintained by the U.S. Army Corps of Engineers for records up to 2006 and found no records of ice jams on the Lower Colorado River (USACE, n.d.-a, accessed February 7, 2007). The licensee performed another search of the database in 2012, which also did not reveal any recorded ice jam events from 2006 to 2012 (USACE, n.d.-a, accessed August 1, 2012).

As part of its review, the staff performed a search of the "Ice Jam Database" and found no records of ice jams on the Lower Colorado River (USACE, n.d.-a, accessed October 11, 2012). Additionally, staff queried water temperatures available at the LCRA website.
The lowest recorded water temperature at LCRA site number 12284 (Bay City) from February 9, 1984 to June 11, 2013 was 43.7 °F (LCRA, n.d.).

The staff confirmed the licensee's conclusion that the reevaluated hazard for ice-induced flooding of the site is bounded by the current design-basis flood hazard.

2.9 Channel Migrations or Diversions

The licensee reported in its FHRR that the reevaluated hazard, including associated effects, for site flooding from channel migrations or diversions does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The staff describes its evaluation of site flooding from channel migrations or diversions, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

The licensee evaluated the potential for channel migration or diversion of the Lower Colorado River due to geologic effects, groundwater pumping, floods, coastal storm surge, mining, and human-induced effects in the STP Units 3 and 4 FSAR Section 2.4S.9 (STPNOC, 2012a). The licensee's evaluation is summarized as follows:

Geologic Effects
STP is located in a region with flat topography, with less than one degree dip in geologic units. Because of low reliefs in the lower Colorado River Basin near the STP site, slope failures are unlikely. Additionally, there is no indication of capable faults in the region near STP. This makes it very unlikely that a seismic mechanism could cause a channel diversion.

Groundwater Pumping
Groundwater pumping can cause land subsidence and consequently diversion of rivers. The measured subsidence in the area near Bay City, TX was 0.12 ft (0.037 m) from 1918 to 1951. Most was attributed to groundwater pumping after 1940. The subsidence increased from 1943 to 1970 to more than 1.5 ft (0.46 m), again due to groundwater withdrawals. Recent withdrawals have declined: 38,554 acre-ft (47,556,000 m^3) was withdrawn in 1980, 37,537 acre-ft (46,301,000 m^3) in 1990, and 14,413 acre-ft (17,778,000 m^3) was withdrawn in 1997.

Flooding
Flooding was reviewed as a possible cause of migration or diversion. The lower Colorado River has been regulated since 1938. The construction of Lake Buchanan and Lake Travis has reduced the peak discharge on the Colorado River near Austin, TX. In September of 1952, a flood occurred which would have generated an estimated flow greater than 20,000 m^3/s 700,000 ft^3/s) at Austin, TX in the absence of regulation. Instead, the recorded peak flow was only 3,720 ft^3/s (105 m^3/s).

Coastal Storm Surge

Coastal storm surge was reviewed as a possible cause of channel migration or diversion. The observed erosion and diversions due to hurricanes in the past 60 years have been repaired by shoreline deposition and wind-blown sediments. Most channels formed by previous storms are closed by existing beaches and are not expected to affect any safety function at STP.

Mining
Gravel mining has caused local diversions and cutoffs due to the presence of abandoned mining pits. Additional concerns have been expressed regarding the potential for gravel mining causing severe downstream bed degradation which may lead to exposed pipelines and bridge failures. However, to this date, there have been no documented cases of severe bed degradation in the Lower Colorado River.

Human-Induced Effects
Human-induced diversions of the Lower Colorado River have been documented as far back as the 17th century. Large log jams were common. The last such jam was partially removed with explosives in 1925, with the remainder moved downstream in 1929 due to a major flood, leading to repeated floods in the Matagorda area. The last major flood occurred in 1935, when the Colorado River almost diverted into Tres Palacios Creek and Tres Palacios Bay. However, since then, dam construction and flood control measures have reduced flooding in the lower Colorado area.

The staff reviewed the licensee's findings and found no evidence of channel diversion. The Lower Colorado River is currently regulated in such a way to make diversion very unlikely. Reservoirs, dams, and levee construction have helped to stabilize the river's course. The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from channel migrations or diversions is bounded by the current design-basis flood hazard.

3.0 INTEGRATED ASSESSMENT AND ASSOCIATED HAZARD DATA

The staff confirmed that the reevaluated hazard results for all reevaluated hazard mechanisms are bounded by the current design-basis flood hazard. Therefore, the staff concludes that an Integrated Assessment is not necessary.

4.0 CONCLUSION

The NRC staff has reviewed the information provided for the reevaluated flood-causing mechanisms of STP, Units 1 and 2. Based on its review, the staff concludes that the licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews.

Based upon the preceding analysis, the NRC staff confirmed that the licensee responded appropriately to Enclosure 2, of the 50.54(f) letter, dated March 12, 2012. In reaching this determination, staff confirmed the licensee's conclusions that (a) the reevaluated hazard results for each reevaluated flood-causing mechanism are bounded by the current design-basis flood hazard, and (b) an Integrated Assessment is not necessary. The NRC staff has no additional information needs at this time with respect to Enclosure 2.

5.0 REFERENCES

Note: ADAMS Accession Nos. refer to documents available through the NRC's Agencywide Document Access and Management System (ADAMS). Publicly-available ADAMS documents may be accessed through http://www.nrc.gov/reading-rm/adams.html.

U.S. Nuclear Regulatory Commission Documents and Publications

NRC (U.S. Nuclear Regulatory Commission), 2007, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition", NUREG-0800, 2007. ADAMS stores the *Standard Review Plan* as multiple ADAMS documents, which are most easily accessed through the web page http://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr0800/.

NRC (U.S. Nuclear Regulatory Commission), 1977, "Design Basis Floods for Nuclear Power Plants," Regulatory Guide (RG) 1.59, Revision 2, August 1977, ADAMS Accession No. ML003740388.

NRC (U.S. Nuclear Regulatory Commission), 2011a, "Near-Term Report and Recommendations for Agency Actions Following the Events in Japan," Commission Paper SECY-11-0093, July 12, 2011, ADAMS Accession No. ML11186A950.

.NRC (U.S. Nuclear Regulatory Commission), 2011b, "Recommendations for Enhancing Reactor Safety in the 21st Century: The Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident," Enclosure to SECY-11-0093, July 12, 2011, ADAMS Accession No. ML111861807.

NRC (U.S. Nuclear Regulatory Commission), 2011c, "Recommended Actions to be Taken Without Delay from the Near-Term Task Force Report," Commission Paper SECY-11-0124, September 9, 2011, ADAMS Accession No. ML11245A158.

NRC (U.S. Nuclear Regulatory Commission), 2011d, "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned," Commission Paper SECY-11-0137, October 3, 2011, ADAMS Accession No. ML11272A111.

NRC (U.S. Nuclear Regulatory Commission), 2011e, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, November 2011, ADAMS Accession No. ML11321A195.

NRC (U.S. Nuclear Regulatory Commission), 2012a, *letter from Eric J. Leeds, Director, Office of Nuclear Reactor Regulations and Michael R. Johnson, Dir*ector, Office of New Reactors, to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, March 12, 2012, ADAMS Accession No. ML12053A340.

NRC (U.S. Nuclear Regulatory Commission), 2012b, letter from Eric J. Leeds, Director, Office of Nuclear Reactor Regulation, to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, May 11, 2012, ADAMS Accession No. ML12097A510.

NRC (U.S. Nuclear Regulatory Commission), 2012c, "Guidance for Performing the Integrated Assessment for External Flooding," Japan Lessons-Learned Project Directorate, Interim Staff Guidance JLD-ISG-2012-05, Revision 0, November 30, 2012, ADAMS Accession No. ML12311A214.

Codes and Standards

NRC (U.S. Nuclear Regulatory Commission), 2012d, "Guidance For Performing A Tsunami, Surge, or Seiche Hazard Assessment," Japan Lessons Learned Directorate Interim Staff Guidance JLD-ISG-2012-06, Revision 3 (Draft), October 18, 2012, ADAMS Accession No. ML12271A036.

NRC (U.S. Nuclear Regulatory Commission), 2012e, South Texas Project, Units 3 and 4 Combined License Application, Safety Evaluation Report (SER) Section 2.4, Rockville, Maryland. ADAMS Accession No. ML12298A415.

NRC (U.S. Nuclear Regulatory Commission), 2013, "Guidance for Assessment of Flooding Hazards Due to Dam Failure," Japan Lessons Learned Directorate Interim Staff Guidance JLD-ISG-2013-01, Revision 3 (Draft), April 17, 2013, ADAMS Accession No ML13057A863.

NRC (U.S. Nuclear Regulatory Commission), 2014, letter from Balwant K. Singal, Senior Project Manager, NRC Plant Licensing Branch IV-1, to STP Nuclear Operating Company dated June 23, 2014, Subject: South Texas Project, Units 1 and 2 – Staff Assessment of the Flooding Walkdown Report Supporting Implementation of Near-Term Task Force Recommendation 2.3 Related to the Fukushima Dai-Ichi Nuclear Power Plant Accident (TAC Nos. MF0283 and MF0284), ADAMS Accession No. ML14150A191.

Codes and Standards

ANS (American Nuclear Society), 1992, *Determining Design Basis Flooding at Power Reactor Sites*, ANSI/ANS-2.8-1992, ANS Standards Committee Working Group ANS-2.8, American Nuclear Society, La Grange Park, Illinois.

Other References

Bailey, J., Shrestha, P., Dawson, C., and Head, S., 2014. "Analysis of Maximum Probable Storm Surge at the South Texas Project Site," In *World Environmental and Water Resources Congress 2014: Water Without Borders*, Conference proceedings, Portland, OR, June 1-5, 2014. Ed. Wayne C. Huber., American Society of Civil Engineers, pp. 1412-1421.

Burcharth, H.F., and Hughes, S.A., 2011, "Fundamentals of Design", in *Coastal Engineering Manual*, Part VI, Design of Coastal Project Elements, Chapter VI-5, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

Cornell (Cornell University), n.d., "COMCOT: A Tsunami Modeling Package," (Web site for the Cornell Multi-grid Coupled Tsunami Model), http://ceeserver.cee.cornell.edu/pll-group/comcot.htm, accessed July 8, 2014.

Demirbilek, Z. and Vincent, C.L., 2008, "Water Wave Mechanics", in *Coastal Engineering Manual*, Part II, Coastal Hydrodynamics Chapter II-1, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

Donnell, B.P., Letter, J.V., McAnally, W.H., and Thomas, W.A., 2008, "User's Guide for RMA Version 4.5." U.S. Army, Engineer Research and Development Center Waterways Experiment Station, Coastal and Hydraulics Laboratory, Vicksburg, MS.

FERC (Federal Energy Regulatory Commission), 1991, "Embankment Dams", in *Engineering Guidelines for the Evaluation of Hydropower Projects*, Chapter 4, Washington, D.C. Accessed November 7, 2013 at http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide.asp.

FERC (Federal Energy Regulatory Commission), 2001, "Determination of the Probable Maximum Flood", in *Engineering Guidelines for the Evaluation of Hydropower Projects*, Chapter 8, Washington, D.C. Accessed November 7, 2013 at http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide.asp.

Fread, D.L., 1991, "BREACH: An Erosion Model for Earthen Dam Failures", Revision 1, Hydrologic Research Laboratory, National Weather Service, NOAA, Silver Spring, Maryland. [No online source]

Fread, D.L. and Lewis, J.M., 1998, "NWS FLDWAV Model: Theoretical Description and User Documentation", Hydrologic Research Laboratory Office of Hydrology, National Weather Service, NOAA, Silver Spring, Maryland. [No online source]

Halff (Halff Associates, Inc.), 2002, "Colorado River Flood Damage Evaluation Project-Phase 1", Volume I and Volume II, prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers. Fort Worth, Texas.

Hansen, E.M., Schreiner, L.C., and Miller, J.F., 1982, *Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian*, NOAA Hydrometeorological Report No. 52, U.S. Department of Commerce, National Oceanographic and Atmospheric Administration, National Weather Service, Washington, D.C. Available online at http://www.nws.noaa.gov/oh/hdsc/PMP_documents/HMR52.pdf.

HCFCD (Harris County Flood Control District), n.d., "GIS: Putting Geographic Information Systems to Work For You," (Web page; title is as it appeared when re-accessed July 8, 2014), http://www.hcfcd.org/gis.asp?flash=yes, accessed 2002.

Knight, W., 2006, "Model Predictions of Gulf and Southern Atlantic Coast Tsunami Impacts from a Distribution of Sources," *Science of Tsunami Hazards*, vol. 24, pp. 304-312.

LCRA (Lower Colorado River Authority), n.d., "LCRA Water Quality Data," web page, http://waterquality.lcra.org/.

Lockhart, J.H., Jr., and Morang, A., 2002, "History of Coastal Engineering", in *Coastal Engineering Manual*, Part I, Chapter I-3, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

Mader, C.L., 2001, "Modeling the 1755 Lisbon Tsunami," *Science of Tsunami Hazards*, vol. 19, pp. 93-98.

NOAA (National Oceanic and Atmospheric Administration),1979, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States," NOAA Technical Report NWS 23, NOAA, National Weather Service, Washington DC, September 1979. Available online at http://docs.lib.noaa.gov/noaa_documents/NWS/TR_NWS/TR_NWS_23.pdf.

NOAA (National Oceanic and Atmospheric Administration), 2013, "State Climate Extremes Committee – Records." Available online at http://www.ncdc.noaa.gov/extremes/scec/records, accessed November 5, 2013.

NOAA (National Oceanic and Atmospheric Administration), n.d.-a, "Center for Operational Oceanographic Products and Services," http://tidesandcurrents.noaa.gov/, accessed 2008.

NOAA (National Oceanic and Atmospheric Administration), n.d.-b, "ETOPO5 5-minute gridded elevation data," http://www.ngdc.noaa.gov/mgg/global/etopo5.HTML, accessed July 8, 2014.

NRCS (Natural Resource Conservation Service), 1986, *Urban Hydrology for Small Watersheds*, Technical Release 55, U. S. Department of Agriculture, Natural Resources Conservation Service, Washington, D.C. Available online at http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf.

Pope, J., and Lockhart, J.H., Jr. 2003, "The Coastal Engineering Manual," in *Coastal Engineering Manual*, Part I, Introduction Chapter I-4, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

Resio, D.T., Bratos, S.M., and Thompson, E.F., 2008, "Meteorology and Wave Climate", in *Coastal Engineering Manual*, Part II, Chapter II-2, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

Scheffner, N.W., 2008, "Water Levels and Long Waves", in *Coastal Engineering Manual*, Part II, Chapter II-5, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?.

Schreiner, L.C., and Reidel, J. T., 1978, *Probable Maximum Precipitation Estimates, United States, East of the 105th Meridian*, Hydrometeorological Report No. 51, U.S. Department of Commerce, National Oceanographic and Atmospheric Administration, National Weather Service, Silver Spring, Maryland. Available online at http://www.nws.noaa.gov/oh/hdsc/PMP_documents/HMR51.pdf.

Smith J. M., and Sherlock, A.R., 2007. "Full-Plane STWAVE with Bottom Friction: II. Model Overview," System-Wide Water Resources Program Technical Note, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Smith, J.M., Sherlock, A.R., and Resio, D.T., 2001. STWAVE: Steady-State Spectral Wave Model User's Manual for STWAVE, Version 3.0," ERDC/CHL SR-01-1. U.S. Army Engineer Research and Development Center, Vicksburg MS.

Sorensen, R., and Thompson, E.F., 2008, "Harbor Hydrodynamics," in *Coastal Engineering Manual*, Part II, Coastal Hydrodynamics Chapter II-7, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, DC. Available online at http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?

STPNOC (South Texas Project Nuclear Operating Company), 2010, "South Texas Project, Units 1 and 2, Updated Final Safety Analysis Report, Revision 15, Chapter 2, Site Characteristics," April 29, 2010, ADAMS Accession No. ML103220294 (Non-Public). (Transmittal memo for this report, dated April 29, 2010, at ADAMS Accession No. ML103220222).

STPNOC (South Texas Project Nuclear Operating Company), 2012a, “South Texas Project Units 3 and 4 Combined License Application, Final Safety Analysis Report”, Revision 7, February 1, 2012, ADAMS Accession No. ML12048B259 (document package). (See ADAMS Accession Nos. ML12048A985 andML12048A998 in this document package for Chapter 2, Site Characteristics.)

STPNOC (South Texas Project Nuclear Operating Company), 2012b, “Flooding Walkdown Summary Report For South Texas Project Units 1 &2,” Attachment 1, NOC-AE-12002932, November 26, 2012, ADAMS Accession No. ML13010A474 (Not Publicly Available).

STPNOC (South Texas Project Nuclear Operating Company), 2012c, Letter from D.W. Rencurrel (STPNOC) to NRC dated June 5, 2012, Subject: Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, ADAMS Accession No. ML12163A127.

STPNOC (South Texas Project Nuclear Operating Company), 2013, Letter to NRC from G. T. Powell dated March 11, 2013, Subject: “South Texas Project Units 1 & 2, Docket Nos. STN 50-498, STN 50-499, Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 Flooding of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, Enclosure 2, Required Response 2, Flood Hazard Reevaluation Report,” Wadsworth, Texas. ADAMS Accession No. ML13079A806. (Enclosure containing report is not publicly available.)

STPNOC (South Texas Project Nuclear Operating Company), 2014a, Letter to NRC from G. T. Powell (STPNOC) dated January 22, 2014, Subject: “South Texas Project Units 1 & 2, Docket Nos. STN 50-498, STN 50-499, STPNOC Partial Response to Request for Additional Information Associated With Near-Term Task Force Recommendation 2.1, Flood Hazard Reevaluation,” ADAMS Accession No. ML14041A336.

STPNOC (South Texas Project Nuclear Operating Company), 2014b, Letter to NRC from G. T. Powell (STPNOC) dated February 13, 2014, Subject: “South Texas Project Units 1 & 2, Docket Nos. STN 50-498, STN 50-499, STPNOC Second Partial Response to Request for Additional Information Associated With Near-Term Task Force Recommendation 2.1, Flood Hazard Reevaluation,” ADAMS Accession No. ML14056A195.

ten Brink, U.S., Twichell, D., Geist, E.L., Chaytor, J., Locat, J., Lee, H., Buczkowski, B., Barkan, R., Solow, A.R., Andrews, B.D., Parsons, T., Lynett, P., Lin, J., and Sansoucy, M., 2008, *Evaluation of Tsunami Sources with the Potential to Impact the U.S. Atlantic and Gulf Coasts: An Updated Report to the Nuclear Regulatory Commission*, U.S. Geological Survey Administrative Report, 302 pp.

TNRIS (Texas Natural Resources Information System), n.d., http://www.tnris.org/, accessed July 8, 2014.

Trabant, P., Watts, P., Lettieri, F. and Jamieson, G., 2001, “East Breaks Slump, Northwest Gulf of Mexico,” Offshore Technology Conference, Houston, TX, April 30, 2001, OTC paper 12960.

TUDelft (Delft University of Technology), n.d., “SWAN,” (Web page for the SWAN (Simulating Waves Nearshore) model), http://www.swan.tudelft.nl/, accessed July 8, 2014.

TWDB (Texas Water Development Board), n.d., “GIS Data,” (Web page for downloading data sets), http://www.twdb.state.tx.us/mapping/gisdata.asp, accessed 2007.

USACE (U.S. Army Corps of Engineers), 2010a, "Hydrologic Modeling System HEC-HMS, Version 3.5", U. S. Army Corps of Engineers, Institute for Water Resources, Hydrologic Engineering Center, Davis, California. Available online at http://www.hec.usace.army.mil/software/hec-hms/.

USACE (U.S. Army Corps of Engineers),1980, *HEC-2 Water Surface Profiles User's Manual*, US Army Corps of Engineers Institute for Water Resources, Hydrologic Engineering Center (HEC), Davis, CA.

USACE (U.S. Army Corps of Engineers), 2010b, "HEC-RAS River Analysis System, Version 4.1," U. S. Army Corps of Engineers, Institute for Water Resources, Hydrologic Engineering Center, Davis, California. Available online at http://www.hec.usace.army.mil/software/hec-ras/.

USACE (U.S. Army Corps of Engineers), n.d.-a, "Ice Jam Database", U.S. Army Corps of Engineers, Cold Region Research and Engineering Laboratory (CRREL), Available online at: http://www.crrel.usace.army.mil/technical_areas/hh/, accessed on various dates.

USACE (U.S. Army Corps of Engineers), n.d.-b, "WAM – WAve prediction Model," U.S. Army Corps of Engineers Coastal and Hydraulics Laboratory, http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=software;8, accessed July 8, 2014.

Ward, S. N., and Day, S., 2001, "Cumbre Vieja Volcano - Potential Collapse and Tsunami at La Palma, Canary Islands," *Geophysical Research Letters*, vol. 28, no. 17, pp. 3397-3400.

6.0 ACRONYMS

ADAMS	Agencywide Documents Access and Management System
ADCIRC	Advanced Circulation Model
ANS	American Nuclear Society
CFR	*Code of Federal Regulations*
CLB	current licensing basis
COL	combined license
COLA	combined license application
CRREL	Cold Region Research and Engineering Laboratory
DBF	design-basis flood
DNC	digital nautical charts
ECP	essential cooling pond
ECWIS	essential cooling water intake structure
ECWS	essential cooling water system
FERC	Federal Energy Regulatory Commission
FHRR	Flooding Hazard Reevaluation Report
FSAR	Final Safety Analysis Report
HEC	Hydrologic Engineering Center (of USACE)
HEC-2	Hydrologic Engineering Center model 2
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
HHA	hierarchical hazard analysis
HMR	hydrometeorological report
ISG	interim staff guidance
JPM	joint probability method
LCRA	Lower Colorado River Authority
LCRB	Lower Colorado River Basin
MCR	main cooling reservoir
MSL	mean sea level
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NOS-CO-OPS	National Ocean Service Center for Operational Oceanographic Product Services
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resource Conservation Service
NSC	Nuclear Support Center
NTTF	Near-Term Task Force (NRC)

NWS	National Weather Service
PMF	probable maximum flood
PMH	probable maximum hurricane
PMMW	probable maximum meteorological wind
PMSS	probable maximum storm surge
PMP	probable maximum precipitation
PMT	probable maximum tsunami
SRP	Standard Review Plan
SSC	structures, systems, and components
STP	South Texas Project
STWAVE	STeady State spectral WAVE
STWAVE/WAM	STeady State spectal WAVE / WAve prediction Model
SWG	Southwest Galveston District (of USACE)
TR	Technical Release
UFSAR	Updated Final Safety Analysis Report
UHS	ultimate heat sink
USACE	U.S. Army Corps of Engineers

Table 2.2-1. Flood-Causing Mechanisms and Corresponding Guidance

Flood-Causing Mechanism	SRP Section(s) and JLD-ISG
Local Intense Precipitation and Associated Drainage	SRP 2.4.2 SRP 2.4.3
Streams and Rivers	SRP 2.4.2 SRP 2.4.3
Failure of Dams and Onsite Water Control/Storage Structures	SRP 2.4.4 JLD-ISG-2013-01
Storm Surge	SRP 2.4.5 JLD-ISG-2012-06
Seiche	SRP 2.4.5 JLD-ISG-2012-06
Tsunami	SRP 2.4.6 JLD-ISG-2012-06
Ice-Induced	SRP 2.4.7
Channel Migrations or Diversions	SRP 2.4.9

The SRP is the Standard Review Plan (NRC, 2007). Interim Staff Guidance's are: JLD-ISG-2012-06 (NRC, 2012d); JLD-ISG-2013-01 (NRC, 2013).

Table 3.0-1. Summary of Controlling Reevaluated Flood-Causing Mechanisms

Reevaluated Flood-Causing Mechanisms and Associated Effects that May Exceed the Powerblock Elevation*	ELEVATION, NGVD29
Local Intense Precipitation and Associated Drainage	33.0 ft (10.1 m)
Failure of Dams and Onsite Water Control/Storage Structures	44.5 ft to 50.8 ft (13.6 m to 15.5 m)
Storm Surge	29.3 ft (8.93 m)

*Flood height and associated effects as defined in JLD-ISG-2012-05 (NRC, 2012c).

Table 3.1.2-1. Design-basis Flood Hazards

Flooding Mechanism	Stillwater Elevation NGVD29	Associated Effects NGVD29	Current Design-basis (CDB) Flood Elevation NGVD29	Reference
Local Intense Precipitation and Associated Drainage	32 ft (9.8 m)	NA	32 ft (9.8 m)	FHRR Table 1.2-1
Streams and Rivers	29.0 ft (8.84 m)	Not discussed in CDB	29.0 ft (8.84m)	FHRR Table 1.2-1
Failure of Dams and Onsite Water Control/Storage Structures	13.6 to 15.5 m (44.5 to 50.8 ft, depending on location)	Not discussed in CDB	13.6 to 15.5 m (44.5 to 50.8 ft, depending on location)	FHRR Table 1.2-1
Storm Surge	26.74 ft (8.150 m)	Not discussed in CDB	26.74 ft (8.150 m)	FHRR Table 1.2-1
Seiche	NA	Not discussed in CDB	Not discussed in CDB	Section 2.5
Tsunami	NA	Not discussed in CDB	Not discussed in CDB	Section 2.6
Ice-Induced	NA	Not discussed in CDB	Not discussed in CDB	Section 2.7
Channel Migrations or Diversions	NA	Not discussed in CDB	Not discussed in CDB	Section 2.8

Table 3.2-1. PMP Depths

PMP Duration	Drainage Area	PMP depth, in inches (cm)			
		FHRR Table 2.1-1[a]		FSAR Units 3 and 4, Revision 7[b]	
72 hr	10 mi^2 (25.9 km^2)	55.7	(141.5)	55.7	(141.5)
48 hr	10 mi^2 (25.9 km^2)	51.8	(131.6)	51.8	(131.6)
24 hr	10 mi^2 (25.9 km^2)	47.1	(119.6)	47.1	(119.6)
12 hr	10 mi^2 (25.9 km^2)	38.7	(98.3)	37.8	(96.0)
6 hr	10 mi^2 (25.9 km^2)	32.0	(81.3)	32.0	(81.3)
3 hr		29.7	(75.4)	29.7	(75.4)
2 hr		26.6	(67.6)	26.6	(67.6)
1 hr		19.8	(50.3)	19.8	(50.3)
30 min	Point	14.5	(36.8)	14.5	(36.8)
15 min	Point	9.9	(25.1)	9.9	(25.1)
5 min	Point	6.4	(16.3)	6.4	(16.3)

(a) Source: STPNOC, 2013, Table 2.1-1.
(b) Source: STPNOC, 2012, Table 2.4S.2-4.
Note that 12-hour PMP depths differ between the sources.

Table 3.2-2. STP Unit 1 and 2 Subbasin Drainage Areas

Subbasin	Description	Drainage Area, in mi^2 (km^2)
STP1	Drains toward Kelly Lake	1.099 (2.846)
STP2a	Drains toward Kelly Lake	0.619 (1.603)
STP2b	Drains toward Kelly Lake	0.108 (0.280)
STP3a	Drains toward Kelly Lake	0.100 (0.259)
STP3b	Drains toward Kelly Lake	0.017 (0.044)
STP3c	Drains toward Kelly Lake	0.124 (0.321)
STP4a	Drains toward Kelly Lake	0.072 (0.186)
STP4b	Drains toward Kelly Lake	0.054 (0.140)
STP5a	Drains toward Kelly Lake	0.571 (1.479)
STP5b	Drains toward Kelly Lake	0.211 (0.546)
North1	Drains toward Kelly Lake	1.466 (3.797)
North2	Drains toward Kelly Lake	0.298 (0.772)
NorthB	Drains toward Kelly Lake	0.191 (0.495)
SW	Drains into Little Robbins Slough	0.382 (0.989)

Source: STPNOC, 2013, Table 2.1-2.

Table 3.2-3. STP Unit 1 and 2 Subbasin Times of Concentration

Subbasin	Description	25 Percent Reduced Times of Concentration, in hours
STP1	Drains toward Kelly Lake	0.59
STP2a	Drains toward Kelly Lake	1.38
STP2b	Drains toward Kelly Lake	0.15
STP3a	Drains toward Kelly Lake	0.73
STP3b	Drains toward Kelly Lake	0.21
STP3c	Drains toward Kelly Lake	0.53
STP4a	Drains toward Kelly Lake	0.71
STP4b	Drains toward Kelly Lake	0.91
STP5a	Drains toward Kelly Lake	0.99
STP5b	Drains toward Kelly Lake	0.98
North 1	Drains toward Kelly Lake	3.98
North 2	Drains toward Kelly Lake	3.53
North B	Drains toward Kelly Lake	0.97
SW	Drains into Little Robbins Slough	0.96

Source: STPNOC, 2013, Table 2.1-3.

Table 3.2-4. STP Unit 1 and 2 HEC-HMS Element PMP Peak Discharge

HEC-HMS Element	Drainage Area, in mi^2 (km^2)		Peak Discharge, in ft^3/s (m^3/s)	
Kelly Lake	4.9297	(12.7679)	24,878.8	(704)
North 1	1.4660	(3.7969)	7,975.8	(226)
North 2	0.2980	(0.7718)	1,773.1	(50.2)
North B	0.1905	(0.4934)	2,510.2	(71.1)
R1	3.8308	(9.9217)	22,657.8	(642)
R2	3.1036	(8.0383)	15,892.9	(450)
R3	2.8622	(7.4131)	13,743.5	(389)
R3a	0.1244	(0.3222)	2,177.5	(61.7)
R4	2.7362	(7.0867)	12,399.9	(351)
R5	1.9545	(5.0621)	10,214.4	(289)
R6	0.1905	(0.4934)	2,507.3	(71.0)
STP1	1.0989	(2.8461)	18,690.0	(529)
STP2a	0.6189	(1.6029)	6,712.8	(190)
STP2b	0.1083	(0.2805)	3,007.0	(85.1)
STP3a	0.0996	(0.2580)	1,528.7	(43.3)
STP3b	0.0174	(0.0451)	453.9	(12.9)
STP3c	0.1244	(0.3222)	2,215.4	(62.7)
STP4a	0.0724	(0.1875)	1,129.1	(32.0)
STP4b	0.0536	(0.1388)	729.9	(20.7)
STP5a	0.5706	(1.4778)	7,418.2	(210)
STP5b	0.2111	(0.5467)	2,781.7	(78.8)
US R1	3.8308	(9.9217)	22,657.8	(642)
US R2	3.1036	(8.0383)	15,948.6	(452)
US R3	2.8622	(7.4131)	13,753.4	(389)
US R4	2.7362	(7.0867)	12,421.2	(352)
US R5	1.9545	(5.0621)	10,220.3	(289)
US R6	0.1905	(0.4934)	2,510.2	(71.1)
SW	0.3825	(0.9907)	5,040.2	(143)

Source: STPNOC, 2013, Table 2.1-5.

Table 3.2-5. STP Units 1 and 2 Local Intense Precipitation-Induced PMF Maximum Water-Surface Elevations for HEC-RAS Cross-Sections that Intersect the Power Block Area

River	River Station	Maximum Water-Surface Elevation
STP Drain	13572	33.0 ft (10.1 m)
STP Drain	12990	33.0 ft (10.1 m)
STP Drain	12809	33.0 ft (10.1 m)
South	2824	33.0 ft (10.1 m)
South	2266	32.9 ft (10.0 m)

Source: STPNOC, 2013, Table 2.1-7.

Table 3.4-1. Maximum Water Surface Elevations, in feet (meters) NGVD29, for Unit 1, Unit 2, and NSCMCR Breach Scenarios at Ten Monitoring Locations Shown in Figure 3.4-7

Scenario	Monitoring Locations									
	1	2	3	4	5	6	7	8	9	10
Unit 1	37.7 (11.49)	42.5 (12.95)	37.7 (11.49)	40.0 (12.19)	35.5 (10.82)	35.5 (10.82)	35.4 (10.79)	35.5 (10.82)	34.6 (10.55)	35.4 (10.79)
Unit 2	42.2 (12.86)	37.3 (11.37)	35.9 (10.94)	36.3 (11.06)	35.0 (10.67)	35.4 (10.79)	35.0 (10.67)	35.1 (10.7)	34.5 (10.52)	35.4 (10.79)
NSC	34.8 (10.61)	34.8 (10.61)	35.8 (10.91)	34.3 (10.45)	34.0 (10.36)	33.9 (10.33)	34.0 (10.36)	36.6 (11.16)	33.8 (10.3)	34.0 (10.36)

Source: STPNOC, 2013, Table 2.3-8.

Table 3.4-2. Extrapolated Inundation Period (Exceeding 0.25-ft Depth), in minutes, for Unit 1, Unit 2, and NSCMCR Breach Scenarios at Ten Monitoring Locations Shown in Figure 3.4-7

Scenario	Monitoring Locations									
	1	2	3	4	5	6	7	8	9	10
Unit 1	61.54	55.21	53.60	57.05	58.41	56.25	30.36	40.26	55.73	64.43
Unit 2	54.89	63.96	54.65	64.43	58.41	57.83	30.65	38.43	56.39	65.86
NSC	51.19	53.53	46.21	53.07	47.83	44.70	21.82	39.10	44.65	53.07

Source: STPNOC, 2013, Table 2.3-8.

Table 3.5-1 Staff's Estimate of PMH Parameters

Parameter and Units	Value	Source in NOAA (1979)
Latitude (degrees North)	28.6	
Coriolis parameter f, 1/s	7.1×10^{-5}	
Coastal distance, km (nautical miles)	601.9 (325)	Figures 1.1 and 1.2
Central pressure P_O, cm Hg (in Hg)	66.52 (26.19)	$P_W - \Delta P$
ΔP, cm Hg (in Hg)	9.98 (3.93)	Figure 2.3
Peripheral pressure Pw, cm Hg (in Hg)	76.5 (30.12)	Section 2.2.2
Radius of maximum winds R, km (mi)	8-33.8 (5-21)	Figure 2.5
Forward speed T, m/s (knots)	3.1-10.3 (6-20)	Figure 2.7
Direction (degrees clockwise from North)	85-190	Figure 2.9
Coefficient K	79.5	Figure 2.11
Moving hurricane gradient velocity km/h (mi/h)	253.8 (157.6)	Equation 2.2
Stationary hurricane gradient velocity, km/h (mi/h)	240.8 (149.7)	Equation 2.4

m=meter; ft=foot; s=second; km=kilometer; mi/h=mile per hour; mi=mile; cm=centimeter; in=inch; Hg=mercury

Figure 2.2.4-1 Flood Event Duration

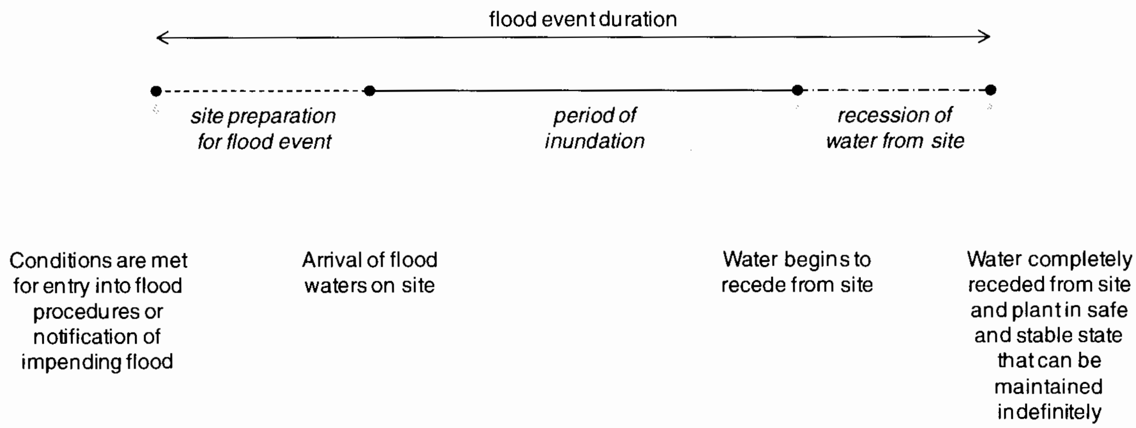

Figure 3.2-1. STP Units 1 and 2 Site, Flow Paths, and Drainage Areas (STPNOC, 2013, Figure 2.1-3)

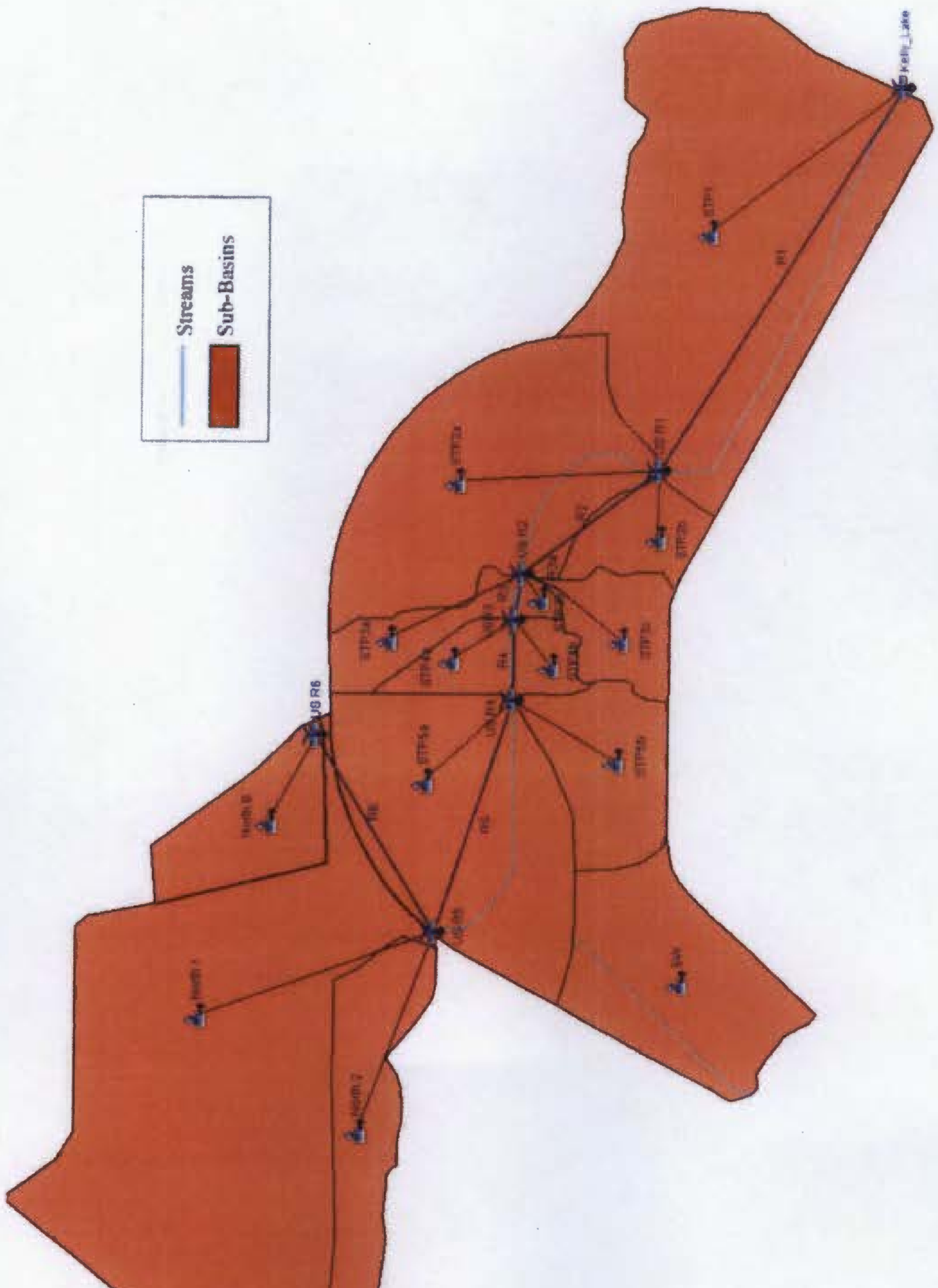
Streams
Sub-Basins
Kelly_Lake
US R6

Figure 3.2-2. HEC-HMS Model Configuration (STPNOC, 2013, Figure 2.1-5)

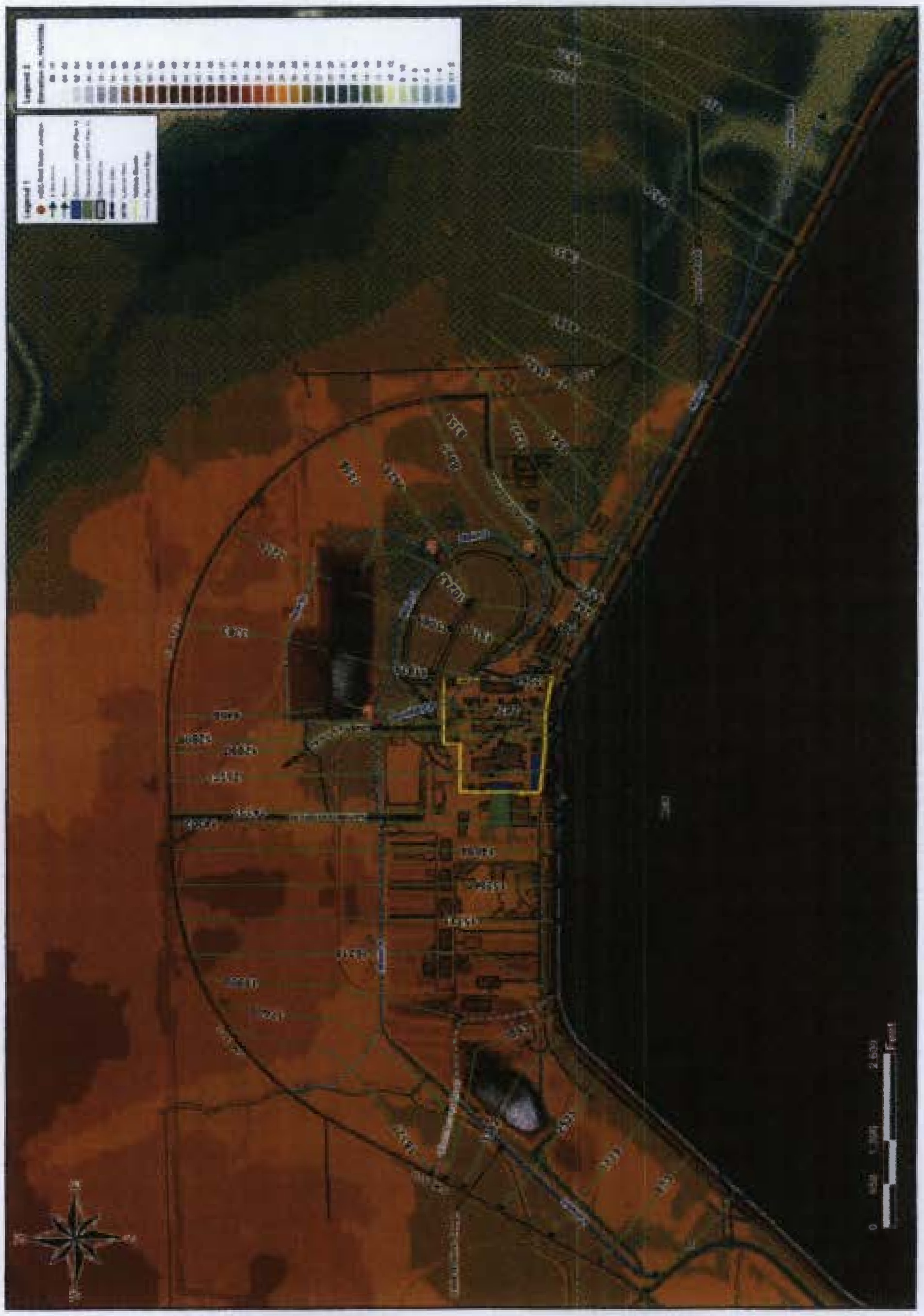

Figure 3.2-3. HEC-RAS Model Cross-Section Locations and Topography (STPNOC, 2013, Figure 2.1-7)

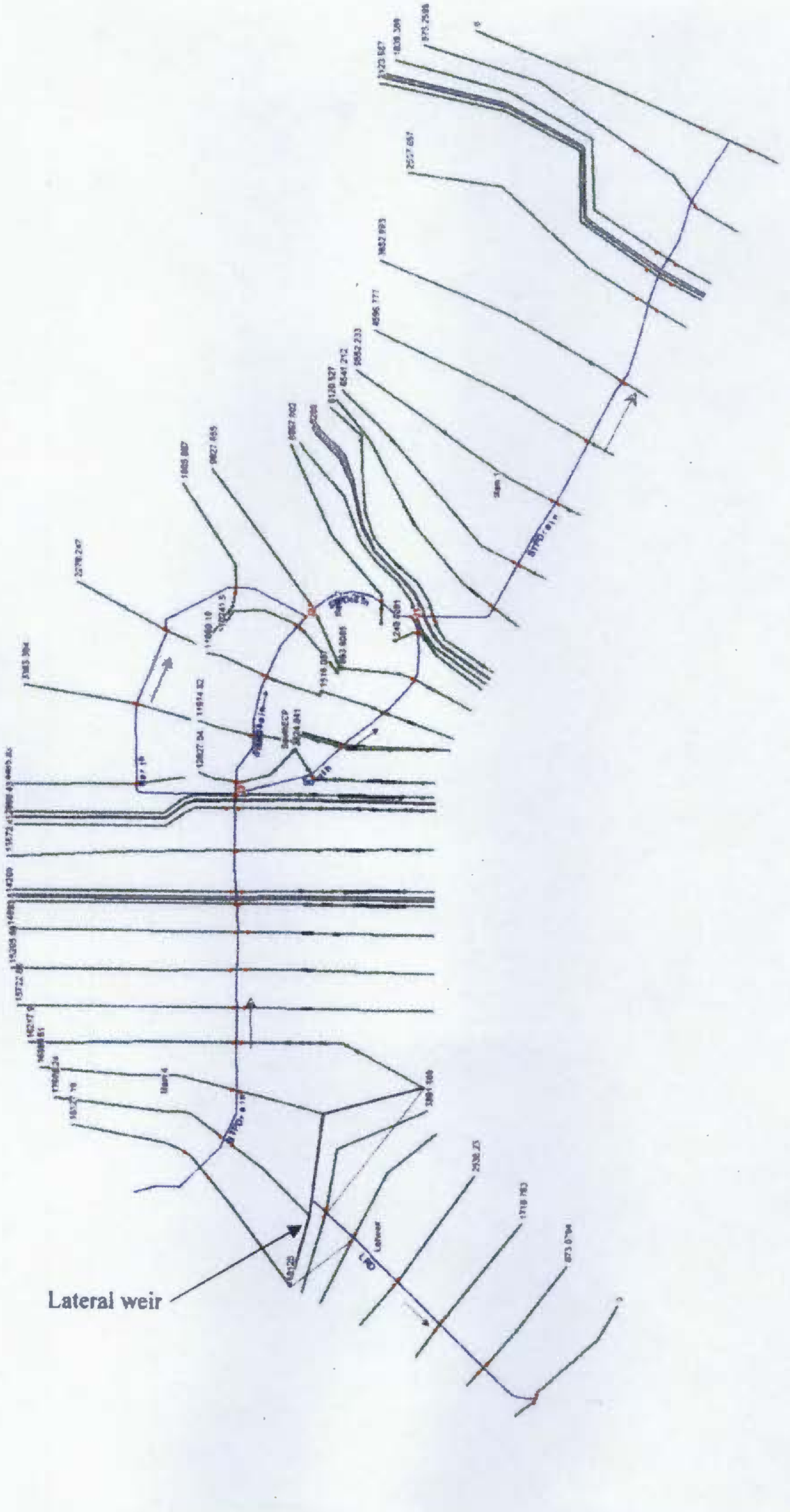
Lateral weir

Figure 3.2-4. HEC-RAS Model Channel Network and Cross Sections Schematic (STPNOC, 2013, Figure 2.1-8)

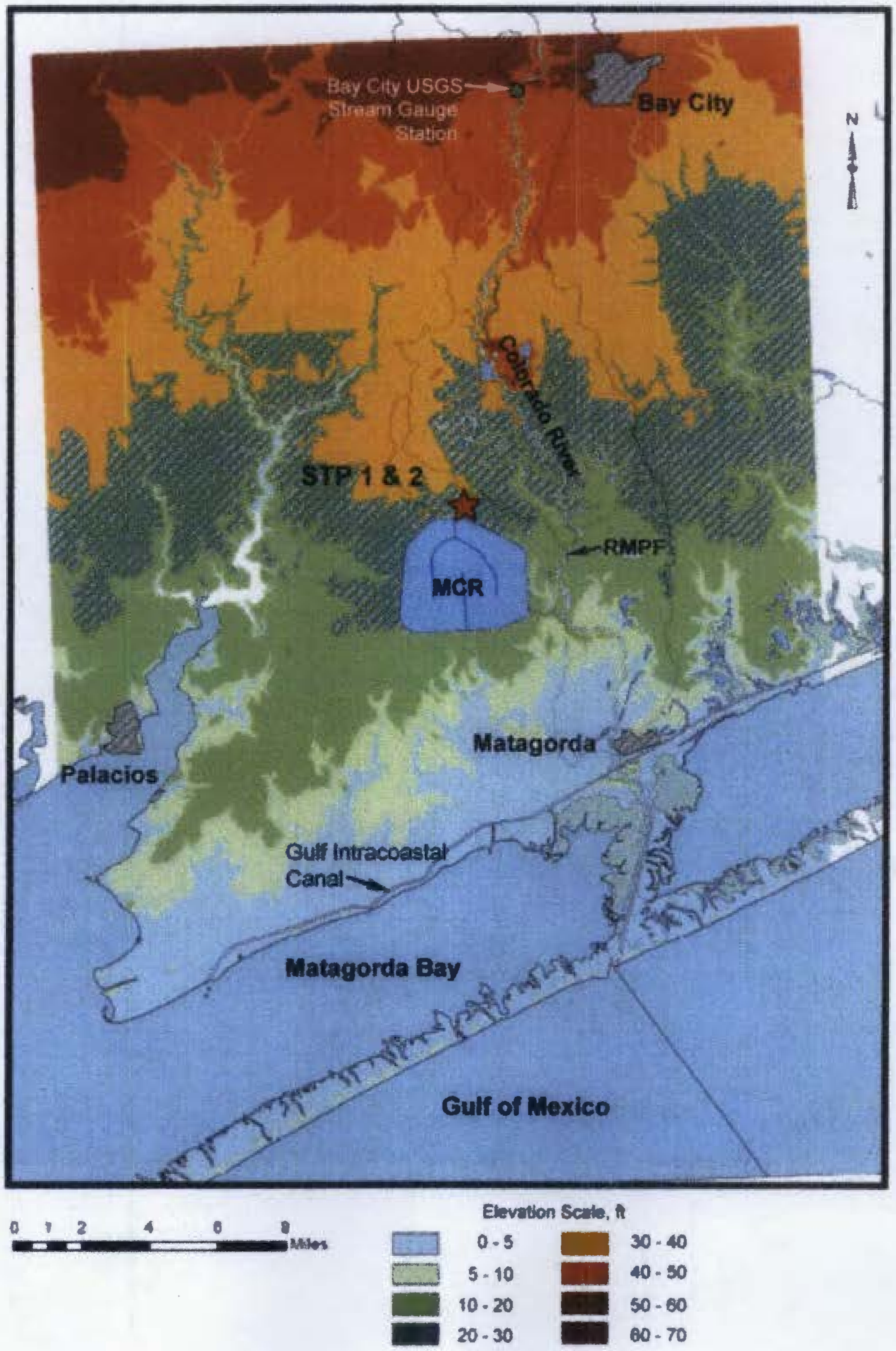

Figure 3.3-1. Location of STP Units 1 and 2 Site in the Lower Colorado River Basin (STPNOC, 2013, Figure 2.2-1a)

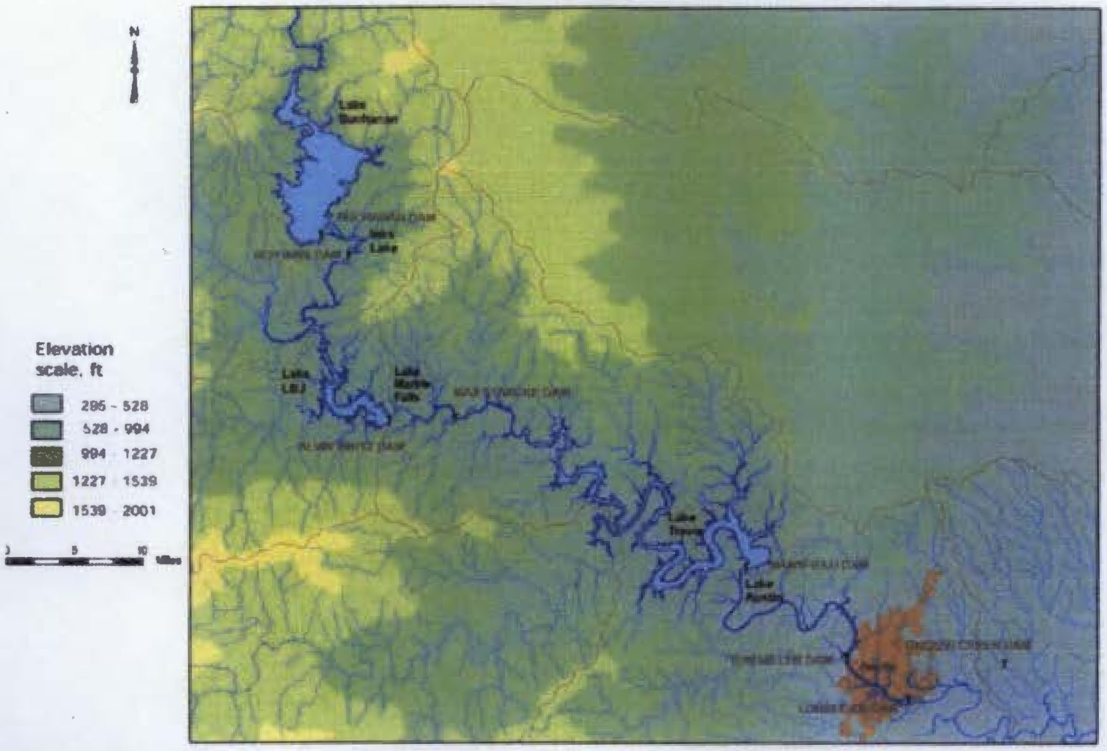

Figure 3.3-2. Hydrologic Features and Dams near the STP Unit 1 and 2 Site (STPNOC, 2013, Figure 2.2-1b)

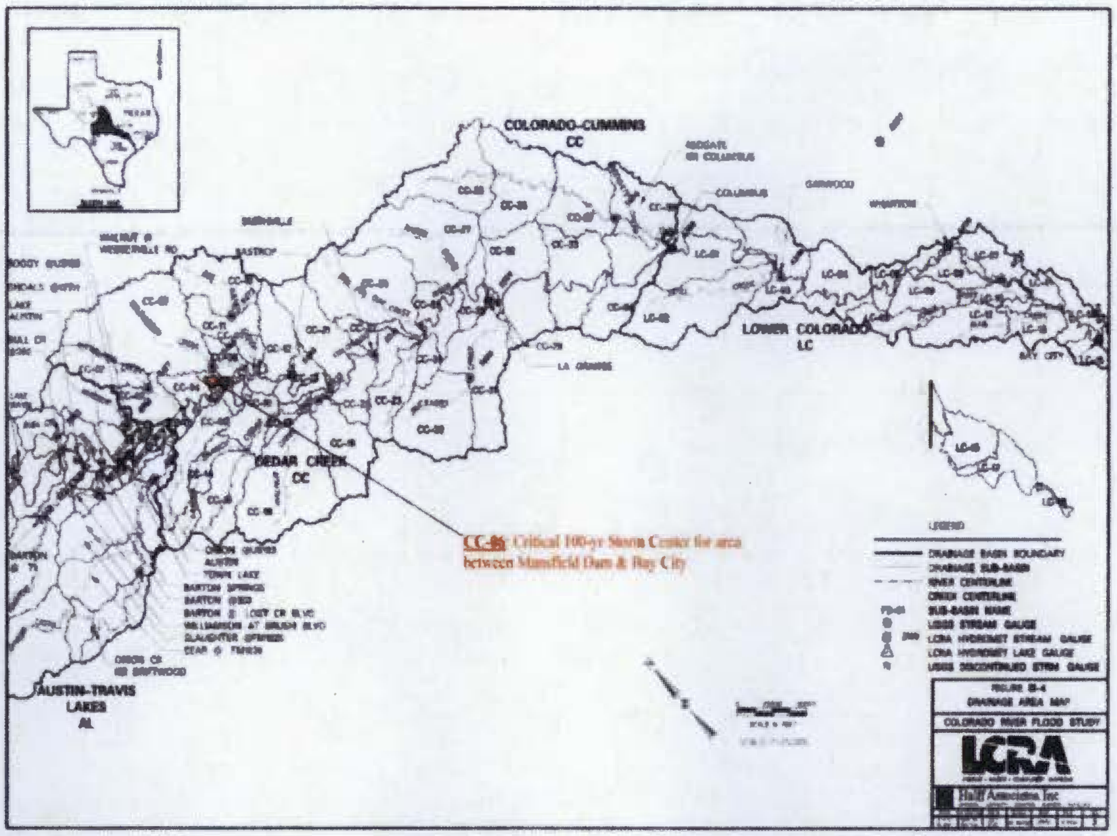

Figure 3.3-3. Subbasins in the Lower Colorado River Basin (STPNOC, 2013, Figure 2.2-2a)

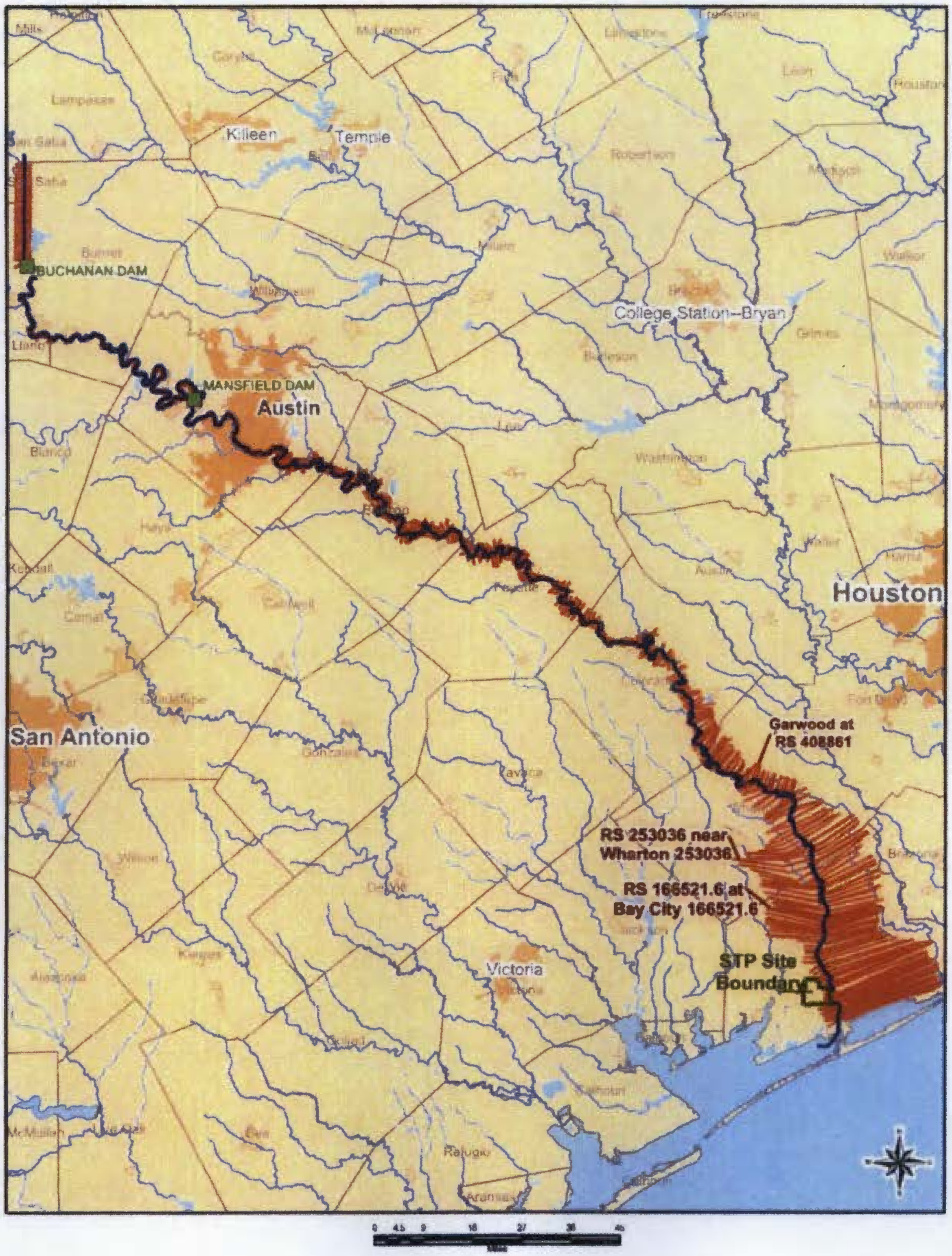
Killeen
Temple
BUCHANAN DAM
College Station--Bryan
MANSFIELD DAM
Austin
Houston
San Antonio
Garwood at
RS 408861
RS 253036 near
Wharton 253036
RS 166521.6 at
Bay City 166521.6
STP Site
Boundary
Victoria

Figure 3.4-1. Locations of Model Cross-Sections in the Dam Break Analysis (STPNOC, 2013, Figure 2.3-4)

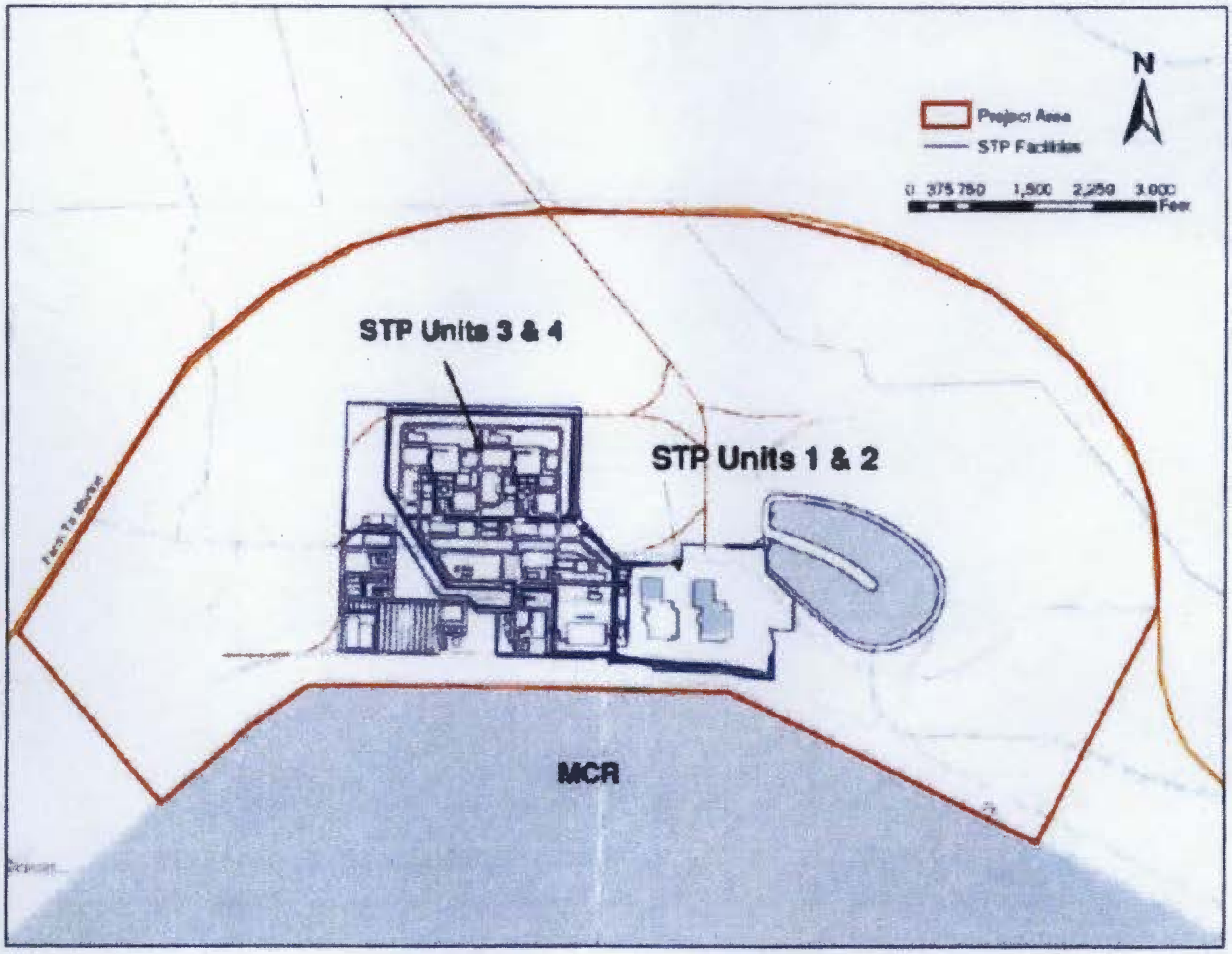

Figure 3.4-2. Location of STP 3 and 4 in Relation to STP 1 & 2 (STPNOC, 2013, Figure 2.3-21)

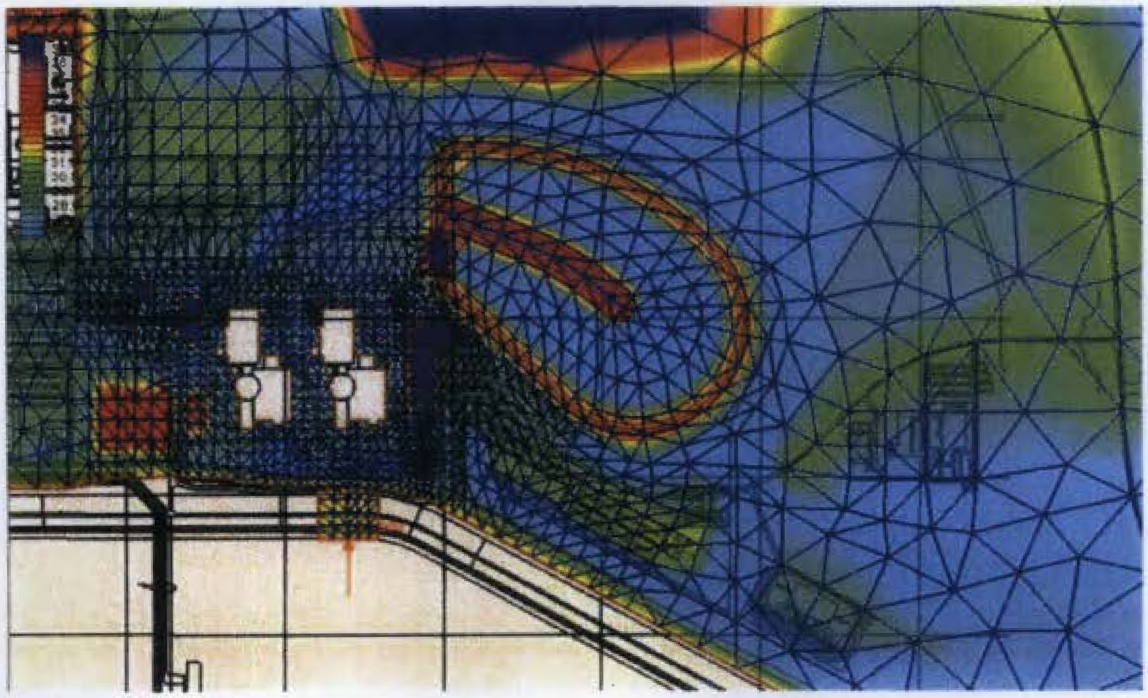

Figure 3.4-3. STP Unit 1-directed Breaching Scenario Location (STPNOC, 2013, Figure 2.3-27a)

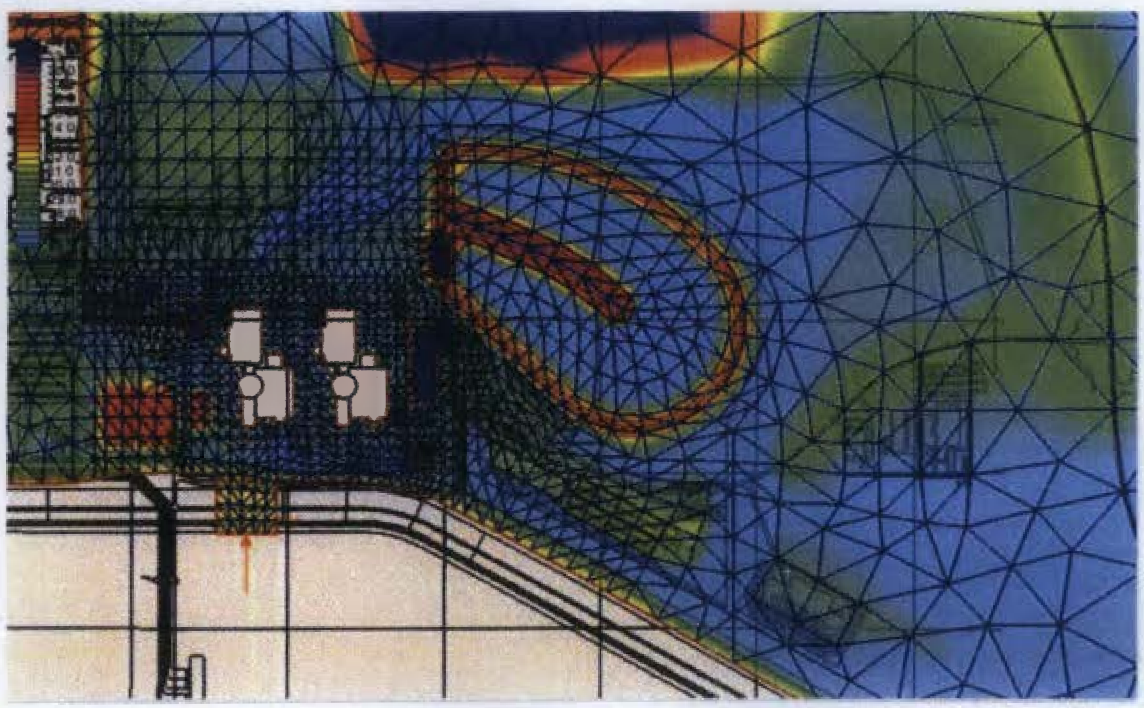

Figure 3.4-4. STP Unit 2-directed Breaching Scenario Location (STPNOC, 2013, Figure 2.3-27b)

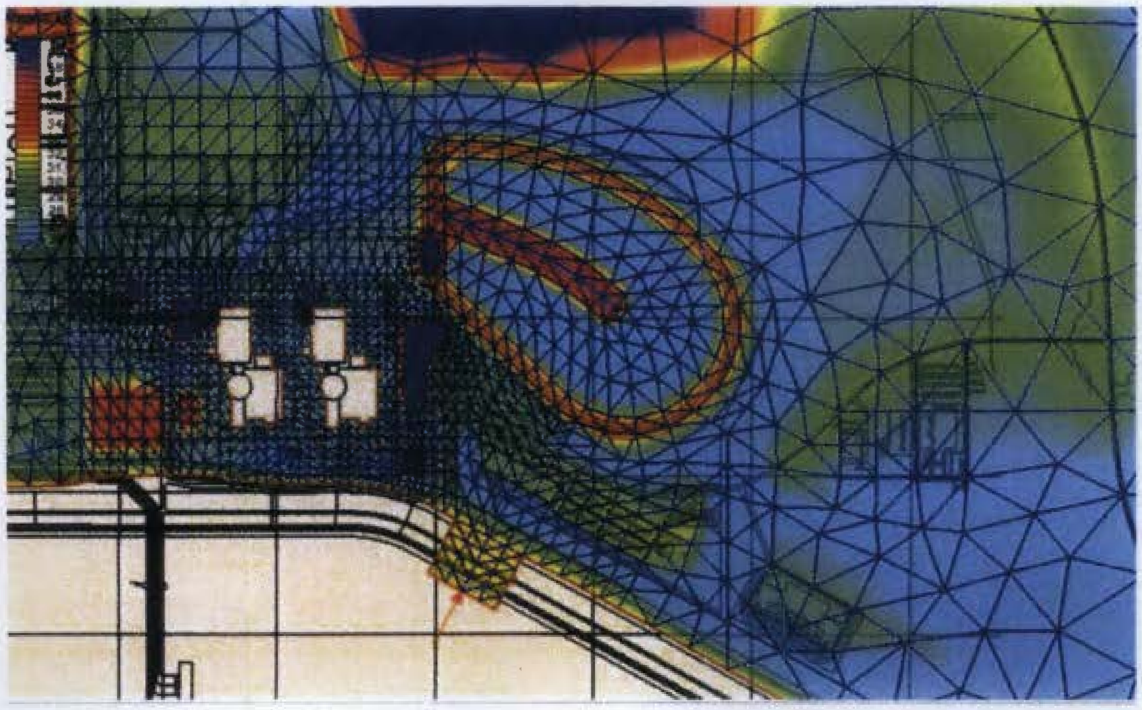

Figure 3.4-5. STP NSC-directed Breaching Scenario Location (STPNOC, 2013, Figure 2.3-27c)

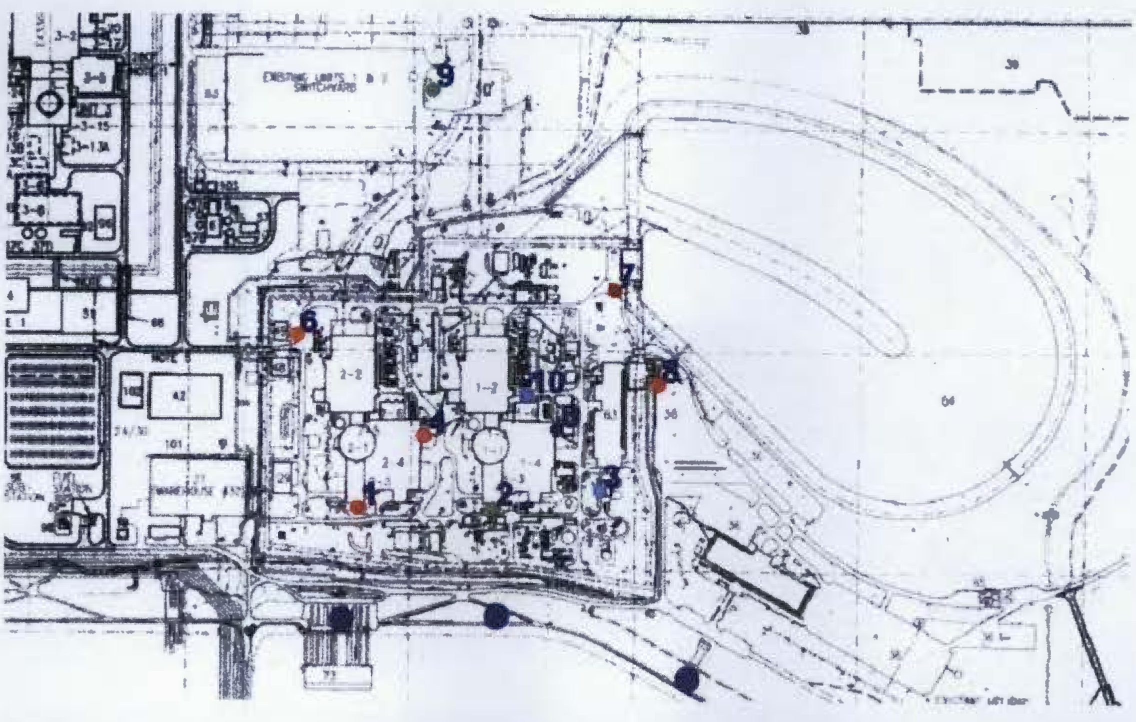

Figure 3.4-6. Location of Breach Monitoring Points 1-10 (STPNOC, 2013, Figure 2.3-28)

September 30, 2014

Mr. Dennis L. Koehl
President and CEO/CNO
STP Nuclear Operating Company
South Texas Project
P.O. Box 289
Wadsworth, TX 77483

SUBJECT: SOUTH TEXAS PROJECT, UNITS 1 AND 2 – STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST FLOOD-CAUSING MECHANISM REEVALUATION (TAC NOS. MF1110 AND MF1111)

Dear Mr. Koehl:

By letter dated March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued a request for information pursuant to Title 10 of the *Code of Federal Regulations*, Section 50.54(f) (hereafter referred to as the 50.54(f) letter). The request was issued as part of implementing lessons-learned from the accident at the Fukushima Dai-ichi nuclear power plant. Enclosure 2 to the 50.54(f) letter requested licensees to reevaluate flood-causing mechanisms using present-day methodologies and guidance.

By letter dated March 13, 2013, STP Nuclear Operating Company responded to this request for the South Texas Project, Units 1 and 2.

The NRC staff has reviewed the information provided and, as documented in the enclosed staff assessment, determined that you provided sufficient information in response to the 50.54(f) letter. This closes out the NRC's efforts associated with TAC Nos. MF1110 and MF1111. If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

Sincerely,
/RA/
Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket Nos. 50-498 and 50-499

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report
cc w/encl: Distribution via Listserv

DISTRIBUTION:

PUBLIC	MMcBride	AKock
JLD R/F	MBensi	BSingal
RidsNroDsea	HJones	RKuntz
RidsNrrDorlLpl4-1	KSee	RidsNrrLASLent
RidsOgcMailCenter	CCook	RidsNrrPMSouthTexas

ADAMS Accession No.: ML14259A195 ***via email**

OFFICE	NRR/JLD/PMB/PM	NRR/JLD/LA*	NRR/DORL/LPLIV-1/PM*
NAME	RKuntz	SLent	BSingal (FLyon for)
DATE	09/18/14	09/17/14	09/30/14
OFFICE	NRO/DSEA/RHMB/BC	JLD/JHMB/BC	NRR/JLD/PMB/PM
NAME	CCook	SWhaley	RKuntz
DATE	09/22/14	09/29/14	09/30/14

OFFICIAL RECORD COPY