CONTENTS

ACRONYMS

2DH	2D-horizontal
ABWR	Advanced Boiling-Water Reactor
ac	acre(s)
ac-ft	acre-feet
ADCIRC	Advanced Circulation
ANS	American Nuclear Society
ANSI	American National Standards Institute
BGS	below ground surface
BTP	Branch Technical Position
CFR	Code of Federal Regulations
CFRW	Crane Foundation Retaining Wall
cfs	cubic (foot) feet per second
COL	combined license
COLA	combined license application
COMCOT	Cornell Multigrid Coupled Tsunami Model
CPGCD	Coastal Plains Groundwater Conservation District
CWS	circulating-water system
d	day(s)
DCD	design control document
DBF	design-basis flood
ECL	effluent concentration limit
ECP	essential cooling pond
EOP	emergency operating procedure
ER	Environmental Report
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FM	Farm-to-Market
FSAR	Final Safety Analysis Report
ft	(foot) feet
gal	gallon(s)
g/cc	grams per cubic centimeter
GHB	general head boundary (condition)
gpd/ft^3	gallon(s) per day per cubic foot
gpm	gallon(s) per minute
HD	horizontal dimension

HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
H:V	horizontal versus vertical
HMR	Hydrometeorological Report
hr	hour(s)
in.	inch(es)
kg	kilogram(s)
kg/m^3	kilogram(s) per cubic meter
km	kilometer
km^2	square kilometer(s)
L	liter(s)
Lb	pound(s)
lb/ft^3	pound(s) per cubic foot
LCW	Low Conductivity Waste
LCWPR	Lower Colorado Water Planning Region
LIDAR	Light Detection And Ranging
Lpm	liter(s) per minute
Lps	liter(s) per second
LRS	Little Robbins Slough
LWMS	Liquid Waste Management System
m	meter(s)
M	million
mb	millibar(s)
MDC	Main Drainage Channel
Mi	mile(s)
MLW	mean low water
MOM	Maximum of Maximum
MOST	Method of Splitting a Tsunami
Mph	mile(s) per hour
MSL	(above) mean sea level
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NGDC	National Geodetic Data Center
NGVD29	National Geodetic Vertical Datum of 1929
nmi	nautical mile(s)
NOAA	National Oceanic and Atmospheric Administration
NOS-CO-OPS	(NOAA's) National Ocean Service Center for Operational Oceanographic Product Services
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NSW	National Weather Service

PMF	probable maximum flood
PMH	probable maximum hurricane
PMP	probable maximum precipitation
PMSS	probable maximum storm surge
PMT	probable maximum tsunami
PMWS	probable maximum wind storm
PMMW	probable maximum meteorological wind
RAI	request for additional information
RMPF	reservoir makeup pumping facility
RSW	reactor service water
s	second(s)
SER	Safety Evaluation Report
SLOSH	Sea, Lake, and Overland Surges from Hurricanes
SRP	Standard Review Plan
SSC	structures, systems, and components
STP	South Texas Project
TDS	total dissolved solids
TS	technical specification
TWDB	Texas Water Development Board
UFSAR	Updated Final Safety Analysis Report
UHS	ultimate heat sink
USACE	U.S. Army Corps of Engineers
USBR	United States Bureau of Reclamation
USGS	U.S. Geological Survey
WES	Waterways Experiment Station

2.4S HYDROLOGIC ENGINEERING

To ensure that a nuclear power plant or plants can be designed, constructed, and safely operated on an applicant's proposed site and in accordance with the U.S. Nuclear Regulatory Commission (NRC or Commission) regulations, NRC staff evaluated the hydrologic impacts on the proposed site. These impacts include the potential for flooding due to precipitation, riverine, and coastal effects. In addition, the staff reviewed the impacts on the site from groundwater flow, ice, and low water effects. These hydrological impacts determine the design-basis flood of a new nuclear power plant and whether flood protection will be required. In addition the staff addressed the potential for the release of radiological material into ground and surface water.

The staff prepared Sections 2.4S.1 through 2.4S.14 of this Safety Evaluation Report (SER) in accordance with the review procedures described in NUREG–0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, (LWR Edition)," the Standard Review Plan (SRP), using information presented in Section 2.4S of the South Texas Project (STP) Units 3 and 4 combined license (COL) Final Safety Analysis Report (FSAR), which references Revision 4 to the Advanced Boiling-Water Reactor (ABWR) design control document (DCD), applicant responses to staff requests for additional information (RAIs), and available reference materials (e.g., those cited in applicable sections of NUREG–0800).

2.4S.1 Hydrologic Description

2.4S.1.1 Introduction

This section of the FSAR describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provides a topographic map showing any proposed changes to natural drainage features.

This SER section provides a hydrologic description of the following specific review areas: (1) the interface of the plant with the hydrosphere including descriptions of site location, major hydrological features in the site vicinity, characteristics related to surface water and groundwater, and the proposed water supply to the plant; (2) hydrological causal mechanisms that may require special plant design bases or operating limitations with regard to floods and water-supply requirements; (3) current and likely future surface-water and groundwater uses by the plant and water users in the vicinity of the site that may affect the safety of the plant; (4) available spatial and temporal data relevant for the site review; (5) alternate conceptual models of the hydrology of the site that reasonably bound hydrological conditions at the site; (6) potential effects of seismic and non-seismic data on the postulated design bases and how they relate to the hydrology in the vicinity of the site and the site region; and (7) any additional information requirements prescribed within the "Contents of Application" sections of the applicable Subparts of Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52. These areas are reviewed in Sections 2.4S.2 through 2.4S.14.

2.4S.1.2 Summary of Application

In Section 2.4S.1 of the FSAR the applicant describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provides a topographic map showing any proposed changes to natural drainage features.

In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.13 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.13 Hydrologic Description

COL License Information Item 2.13 requires COL applicants to provide a detailed description of all major hydrologic features on or in the vicinity of the site and a specific description of the site and all safety-related elevations, structures, exterior accesses, equipment, and systems from the standpoint of hydrologic considerations.

2.4S.1.3 Regulatory Basis

The relevant requirements of the Commission regulations for the hydrologic description, and the associated acceptance criteria, are described in Section 2.4.1 of NUREG–0800.

The applicable regulatory requirements for identifying the site location and describing the site hydrosphere are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site.

- 10 CFR 100.20(c), as it relates to requirements to consider physical site characteristics in site evaluations.

- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The staff also used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- Regulatory Guide (RG) 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.102, “Flood Protection for Nuclear Power Plants”

2.4S.1.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.1 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to site hydrologic description. The staff’s technical review of this section includes an independent review of the applicant’s information in the FSAR and in the responses to the RAIs.

This section describes the staff’s evaluation of the technical information in FSAR Section 2.4S.1.

COL License Information Item

- COL License Information Item 2.13 Hydrologic Description

The staff reviewed the hydrologic description of the STP site and vicinity. The staff's review of major hydrological features and descriptions of the site and safety-related elevations, structures, exterior accesses, equipment, and systems is summarized below.

2.4S.1.4.1 Site and Facilities

This section describes the location of the proposed site and the major facilities of the proposed plant.

Information Submitted by Applicant

The STP Units 3 and 4 site is on the west bank of the Colorado River, opposite river kilometer 23.5 (mile [mi] 14.6) from the Gulf Coast. The STP site is approximately 49.4 square kilometers (km^2) (12,200 acres [ac]) in size including the main cooling reservoir, which has a surface area of approximately 28.3 km^2 (7,000 ac) (see Figure 2.4S.1-1 below). The elevation of the site varies from approximately 4.6 meters (m) (15 feet [ft]) above mean sea level (MSL) south of the main cooling reservoir to approximately 10.4 m (34 ft) MSL near the north edge of the site.

The main cooling reservoir is a manmade reservoir enclosed by a 20-km (12.4- mi) long earthen embankment. The main cooling reservoir is used as the heat sink in a closed-loop cooling system for normal operation of STP Units 1 and 2 and will be similarly used for STP Units 3 and 4. The main cooling reservoir is not a safety-related facility because it will hold no safety-related water for STP Units 3 and 4. The reservoir makeup pumping facility (RMPF), an intake system located on the west bank of the Colorado River, supplies makeup water to the main cooling reservoir. Existing STP Units 1 and 2 use a smaller reservoir, the 0.19-km^2 (46-ac) essential cooling pond (ECP), as the ultimate heat sink (UHS). STP Unit 3 and Unit 4 will each have a UHS consisting of an engineered concrete structure water-storage basin with a dedicated reactor service water (RSW) pump house and dedicated mechanical draft cooling towers. Onsite groundwater wells would be the primary source of makeup water to the UHS basin with the main cooling reservoir as a secondary backup source.

The design-basis flood for the STP site results from a postulated instantaneous breach of a north segment of the main cooling reservoir embankment and is described in detail in FSAR Section 2.4S.4. The applicant determines the design-basis flood elevation to be 12.2 m (40 ft) MSL, which is higher than the normal plant site grade of 10.4 m (34 ft) MSL for STP Units 3 and 4. Safety-related structures, systems, and components (SSCs) require flood protection, which is described in FSAR Section 2.4S.10.

In accordance with the requirements in Appendix A of 10 CFR Part 52, the applicant compares the STP Units 3 and 4 hydrologic site characteristics with the respective envelopes of the ABWR standard plant site design parameters specified in Section 5.0, Table 5.0 of the referenced ABWR DCD Tier 1. The envelope of the ABWR standard site design parameter for a maximum flood level is 1 ft below the plant grade. Because the design-basis flood level at the STP site is higher than the corresponding ABWR standard site design parameter, the applicant identifies this issue as a departure, STP DEP T1 5.0-1, from the certified design.

NRC Staff's Technical Evaluation

The staff conducted a hydrology site audit from March 25, 2008 through March 27, 2008. The site audit included a visit to (1) the STP site and a tour of the RMPF and the barge canal on the Colorado River; (2) the main cooling reservoir, including intake and outfall locations; (3) the STP Units 3 and 4 power block location; and (4) the Little Robbins Slough (LRS), where it crosses the west access road. The staff observed the (1) general site layout, (2) location of Units 3 and 4 in relation to the location of the main cooling reservoir, (3) relief well system on the main cooling reservoir embankment, (4) surface drains that channel surface runoff and relief well discharge into the Colorado River, and (5) main drainage ditch that the applicant proposes to relocate before the construction of STP Units 3 and 4.

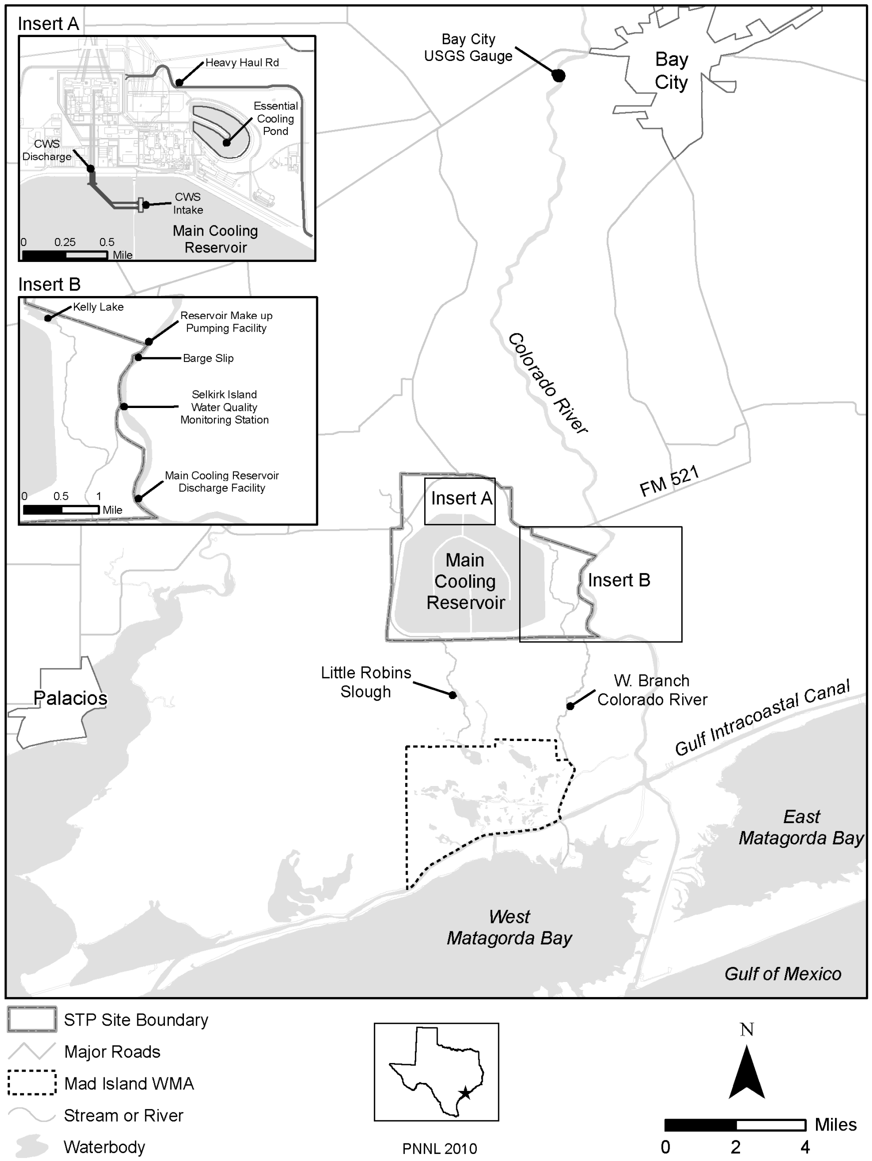

Figure 2.4S.1-1. Map Showing the Location of the STP Site

The staff compared the information from the applicant in FSAR Section 2.4S.1 with publicly available maps and data regarding the STP site and its surrounding region. The STP site is located approximately 14.5 km (9 mi) southwest of Bay City, Texas, and approximately 12.9 km (8 mi) northeast of Palacios, Texas (Figure 2.4S.1-1). The Colorado River flows south on the eastern boundary of the STP site. The West Branch of the Colorado River and the Little Robbins Slough (LRS) are located to the east and west of the main cooling reservoir (Figure 2.4S.1-1). The Matagorda Bay and the Gulf of Mexico are located approximately 19.3 and 24.1 km (12 and 15 mi), respectively, south of the STP Units 3 and 4 location, and the northern tip of Palacios Bay is located about 8 km (5 mi) west of the site.

The staff's evaluation of departure STP DEP T1 5.0-1 is described in SER Sections 2.4S.4 and 2.4S.10.

2.4S.1.4.2 Hydrosphere

This section describes the hydrology in the vicinity of the proposed site, including rivers and streams, lakes and reservoirs, coastal regions, and surface-water and groundwater uses.

Information Submitted by Applicant

The FSAR descriptions of surface water in the vicinity of the STP site include descriptions of the Colorado River Basin, LRS, adjacent drainage basins, shore regions, and surface-water and groundwater uses.

The Colorado River Basin

The Colorado River Basin is 109,603 km^2 (42,318 mi^2) in size, of which 29,534 km^2 (11,403 mi^2) are considered non-tributary. The Upper Colorado River Basin is the portion lying upstream of Lake O.H. Ivie, with an approximate area of 50,857 km^2 (19,636 mi^2). The Lower Colorado River Basin is the remaining portion, 58,746 km^2 (22,682 mi^2) in area, from Lake O.H. Ivie to the Gulf Coast.

The climate of the Colorado River Basin is warm and temperate with dry winters and humid summers. Spring and fall are wet seasons with rainfall peaks in May and September. Convective thunderstorms, typically of short duration and high intensity, dominate spring rainfall. Fall precipitation results from tropical storms and hurricanes that originate in the Caribbean Sea and the Gulf of Mexico. Annual rainfall in the region varies from 112 centimeters (cm) (44 inches [in.]) at the coast to 61 cm (24 in.) inland.

Stream-flow data in the Colorado River Basin have been measured since the early 1900s. There has been a major drought in the basin in almost every decade of the twentieth century. Three major statewide droughts have occurred between 1941 and 1970: from 1947 to 1948, from 1950 to 1957 (the most severe), and from 1960 to 1967.

The Colorado River Basin has 30 dams with individual storage capacities exceeding 12.3 million cubic meters (m^3) (10,000 ac-ft) (FSAR Table 2.4S.1-1). Although the dams in the Colorado River Basin were constructed primarily for flood control, they are also used to supply water. Six of the dams on the Lower Colorado River are operated by the Lower Colorado River Authority (LCRA). These six dams—Buchanan, Inks, Wirtz, Starcke, Mansfield, and Tom Miller—impound the six Highland Lakes: Buchanan, Inks, Lyndon B. Johnson, Marble Falls, Travis, and Austin, respectively. Of these, the Buchanan and Mansfield dams are the two major structures on the Colorado River that may influence flood conditions near the STP site. Both dams were designed or upgraded to safely pass their respective probable maximum floods. Mansfield Dam is currently the most downstream major control structure on the Colorado River and impounds Lake Travis. With a storage capacity of 3,976 million m^3 (3,223,000 ac-ft), Lake Travis is the largest reservoir in the Colorado River Basin. Mansfield Dam and Lake Travis provide most of the floodwater storage capacity in the basin.

Lakes Travis and Buchanan also supply water for communities, industry, irrigation, and aquatic life with water-supply storage capacities of approximately 1,397 and 1,079 million m^3 (1,132,400 and 875,000 ac-ft), respectively.

Wider and flatter lateral slopes characterize the Colorado River flood plain downstream from the city of Columbus compared to the flood plain upstream of the city. The flood plain downstream of the city is also characterized by no discernible valley, and interbasin spillage occurs during high flood discharges.

Downstream of Mansfield Dam are seven U.S. Geological Survey (USGS) stream-flow gauge stations (FSAR Table 2.4S.1-3 and FSAR Figure 2.4S.1-8). The stream-flow gauge closest to the STP site on the Colorado River is located approximately 25.7 km (16 mi) upstream, 3.7 km (2.3 mi) west of Bay City (Texas) at river km 52.3 (river mile 32.5). Stream-flow records at the Bay City gauge have existed since April 1948.

<u>Little Robbins Slough</u>

LRS is an intermittent stream located about 14.5 km (9 mi) northwest of Matagorda, Texas, with a length of approximately 10.5 km (6.5 mi) before it joins Robbins Slough. Robbins Slough is a brackish marsh south of the STP site that flows approximately 6.4 km (4 mi) to the Gulf Intracoastal Waterway. During construction of the main cooling reservoir, LRS was relocated to a channel west of the main cooling reservoir. The relocated LRS flows parallel to the west embankment of the main cooling reservoir and joins its natural course approximately 1 mi east of the southwest corner of the main cooling reservoir.

<u>Adjacent Drainage Basins</u>

The Colorado-Lavaca River Basin is located west of the Colorado River Basin in the coastal region and includes the Tres Palacios Creek, which is not a tributary to the Colorado River or to the Lavaca River. The Colorado-Lavaca River Basin drains into the Tres Palacios Bay. During high flood discharges, such as during the 1931 flood, the floodwaters from the Colorado River overflow the eastern basin ridge into Caney Creek near Wharton, Texas, which is in the San Bernard River Basin. Floodwaters from the Colorado River Basin occasionally spill west into the Colorado-Lavaca Basin.

<u>Shore Regions</u>

The STP site is located approximately 16.9 km (10.5 mi) from Matagorda Bay, approximately 27.2 km (16.9 mi) from the Gulf of Mexico, and approximately 120.7 km (75 mi) from the continental shelf. The Matagorda Peninsula shoreline retreats landward or advances seaward in response to various hydrologic, meteorologic, and climatic factors combined with engineering activities.

The Matagorda Peninsula is a classic microtidal, wave-dominated coastline. The mean diurnal tide varies by approximately 0.6 m (2.1 ft). Based on 20 years of observations, a University of Texas study (Gibeaut et al., 2000) estimated the mean significant wave height of 1 m (3.3 ft) with a mean peak wave period of 5.7 seconds at a location 40 km (24.9 mi) east of the Colorado River entrance in a water depth of 25.9 m (85 ft). Gibeaut et al., (2000) also estimated that the shoreline segment of the Matagorda Peninsula 2.6 km (1.6 mi) southwest of the Colorado River is retreating at a rate of 0.5 to 2.0 m (1.6 to 6.4 ft) per year, whereas the shoreline from this point to the mouth of the river displays a long-term advance. The shoreline northeast of the mouth of the Colorado River only shows a slight long-term advance.

The Colorado River discharged directly into the Gulf through a channel dredged across the Matagorda Peninsula in 1936 after the 1929 removal of a log jam in the Colorado River. In the early 1990s, the U.S. Army Corps of Engineers (USACE) constructed jetties on each side of the river's entrance and dredged an entrance channel. In 1993, the USACE constructed a diversion channel to discharge the Colorado River into the Matagorda Bay. The former river channel is now a navigation channel that connects the IntraCoastal Waterway to the Gulf.

Tropical storms and hurricanes are very common in this region. From 1900 to 2005, 33 major hurricanes of Category 3 and above made landfall on the Texas coast. The applicant states that the expected frequency of occurrence of major hurricanes is approximately once every 3 years.

Surface-Water Use

The Lower Colorado Water Planning Region (LCWPR), or Region K, comprises a total of 15 counties in Texas including Matagorda County, the location of the STP site. Ten aquifer systems and six river and coastal basins form the sources of the water supply to Region K, with the Colorado River representing the largest source of surface water. The Lower Colorado Regional Water Planning Group (2006) estimated the total annual water supply in Region K to be 1,604 million m^3 (1.3 million ac-ft), with a 73-percent contribution from surface-water sources.

The Texas Commission on Environmental Quality maintains a Water Rights Database that contains details of all active and inactive surface-water rights permits and contracts. The LCWPR designates the LCRA and the city of Austin as "wholesale water providers," because they provide a significant amount of water for municipal and manufacturing uses within the region. FSAR Table 2.4S.1-4 lists active surface-water users in Matagorda County. There are no known surface-water users downstream of the STP site.

Groundwater

FSAR Section 2.4S.12 describes local and regional groundwater characteristics, groundwater users, groundwater well locations, and withdrawal rates. Section 2.4S.12.2 of this report summarizes the applicant-provided groundwater-related information.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.1. The staff's independent review and determinations regarding the hydrosphere are described below.

The applicant describes the plant's water demands in Environmental Report (ER) Section 3.3. The UHS system provides water for the safety-related cooling of STP Units 3 and 4. Onsite wells primarily provide makeup water to the engineered UHS basins. During the limited peak demand described in ER Section 3.2, the main cooling reservoir provides supplementary water to the UHS basin, as needed. Surface-water and groundwater sources are not safety-related because the engineered UHS basins of each unit have a sufficient capacity to provide a 30-day cooling-water supply to the UHS without the need for any makeup or blowdown.

It is important to note that the FSAR hydrology sections mostly rely on the National Geodetic Vertical Datum of 1929 (NGVD29) as the referenced vertical datum, and the term MSL is based on the NGVD29. In a few exceptional cases, the applicant uses data referenced in the North American Vertical Datum of 1988 (NAVD88) when referring to a few studies conducted by others. There is a small difference of 0.05 m (0.16 ft) between NGVD29 and NAVD88 near the STP site.

The staff reviewed the applicant's description of the hydrosphere in the vicinity of the site and determined that the description is satisfactory. The staff used the NGVD29-based MSL to reference elevations in this report.

The Colorado River Basin

The Colorado River Basin (Figure 2.4S.1-2) is approximately 109,603 km^2 (42,318 mi^2) in size (LCRWPG, 2006). The Lower Colorado River Basin is the portion downstream of Lake O.H. Ivie. Approximately 90 percent of the contributing area of the basin lies upstream of the Mansfield Dam near Austin, Texas (LCRWPG, 2006). The STP site is located on the west bank of the Colorado River at river kilometer 23.5 (river mile 14.6).

The discharge of the Colorado River near the site is measured at USGS gauge 08162500, near Bay City, Texas. Available stream-flow discharge data at this gauge have been gathered since May 1, 1948.

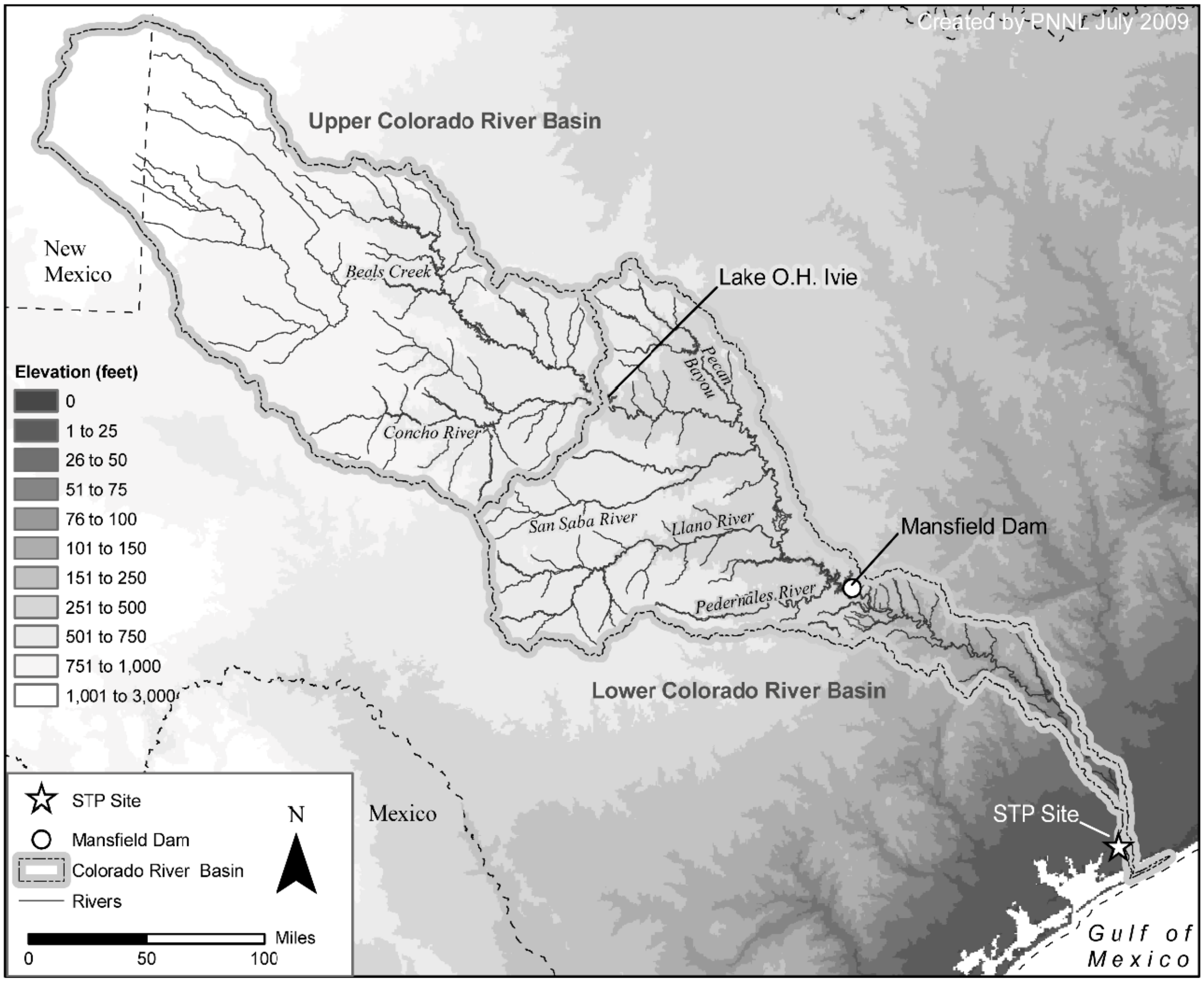

Figure 2.4S.1-2. The Colorado River Basin. (The point of demarcation between the upper and the lower basin is Lake O.H. Ivie. Mansfield Dam and the STP site are also shown.)

The stream flow in the Colorado River downstream of Austin, Texas, is regulated by releases from the Mansfield Dam. The LCRA operates six dams (Buchanan, Inks, Wirtz, Starcke, Mansfield, Tom Miller [LCRA, 2009]) and six respective highland lakes (Buchanan, Inks, Lyndon B. Johnson, Marble Falls, Travis, Austin). Lake Buchanan has a storage capacity of 1,078 million m^3 (875,566 ac-ft) and is used to supply water and to generate hydroelectric power. Lake Travis has a storage capacity of 1,396 million m^3 (1,131,650 ac-ft) and is used to supply water, manage floods, and generate hydroelectric power. The combined water-storage capacity of the six highland lakes is 2,695 million m^3 (2,184,777 ac-ft) (LCRA, 2009). Mansfield Dam provides the most downstream flood-control reservoir in the Colorado River Basin. The broad floodplain in the Lower Colorado Basin has a relatively flat gradient. Interbasin spillage between the Lower Colorado Basin and its adjacent basins can occur during floods because of a lack of steep ridges that separate the subbasins.

Little Robbins Slough

LRS is an intermittent stream that originates approximately 3.2 km (2 mi) northwest of the STP site and has a drainage area of approximately 10.4 km^2 (4 mi^2). During the construction of existing Units 1 and 2 and the main cooling reservoir, the original course of the slough was relocated to the west of the main cooling reservoir. The relocated channel runs along the western edge of the main cooling reservoir embankment, turns east at the southwest corner of the main cooling reservoir embankment, and rejoins its natural course approximately 1.6 km (1 mi) east of the southwest corner of the main cooling reservoir embankment. The LRS flows into Robbins Slough, which is a brackish marsh that joins the Gulf Intracoastal Waterway approximately 6.4 km (4 mi) to the south (Figure 2.4S.1-1). There is no known stream-flow monitoring of the slough.

Adjacent Drainage Basins

The Lower Colorado River Basin is flanked by the Colorado-Lavaca River Basin to the west and the San Bernard Coastal Basin to the east. Flat, wide floodplains and a lack of well-defined basin ridges characterize the terrain near the STP site.

Shore Regions

The Matagorda Peninsula separates the Matagorda Bay from the Gulf, but the southwest portion of the bay is open to the Gulf of Mexico. The shoreline of Matagorda Bay along the Gulf Coast has changed constantly as the result of a combination of hydrologic and meteorological processes, in addition to engineering activities. The shore region of the Matagorda Bay is also affected by waves generated by tropical storms and hurricanes. The hydrologic features of the shore region have also been altered by a series of engineering modifications. After the removal of a log jam on the Colorado River in 1929, the Colorado River directly discharged into the Gulf through a channel dredged across the peninsula in 1936. Beginning in the 1990s, the USACE constructed jetties on each side of the river entrance and dredged an entrance channel. In 1993, the USACE constructed a diversion channel that directs the flow of the Colorado River into the West Matagorda Bay. The former river channel is now a navigation channel connected to the Gulf Intracoastal Waterway. The Gulf Intracoastal Waterway is a 2,090-km (1,300-mi) long manmade canal that runs along the Gulf of Mexico from Brownsville, Texas, to St. Marks, Florida (Texas Department of Transportation 2007).

Surface-Water Use

Water is withdrawn from the Colorado River to support the operations of existing Units 1 and 2 at the STP site. The withdrawn water is used to replace water lost from the main cooling reservoir due to natural and forced evaporation, seepage, and occasional discharge to maintain water quality for the circulating-water systems (CWSs). The main cooling reservoir and the water withdrawal system from the Colorado River will continue to operate as a similar system to support the operations of STP Units 3 and 4. However, water withdrawal from the Colorado River is not a safety-related activity or essential to plant operation, because the engineered UHS has sufficient capacity to operate the plant for 30 days without supplementing its water storage.

The Main Cooling Reservoir

The predominant surface-water feature near the STP site is the main cooling reservoir, a manmade lake impounded by earthen embankments that was constructed on the natural ground surface immediately south of the existing facility. The main cooling reservoir is part of the closed-loop cooling system for STP Units 1 and 2 and acts as the normal heat sink for waste heat generated during the operations of these units. The main cooling reservoir is currently operated to dissipate waste heat from the operations of existing Units 1 and 2, primarily via evaporation, which results in some water loss from the main cooling reservoir. To support the operations of STP Units 1 and 2, the normal maximum water surface elevation of the main cooling reservoir is 14.3 m (47 ft) MSL.

In addition to evaporation, water is lost from the main cooling reservoir due to seepage. About 770 relief wells were installed along the main cooling reservoir embankment during the construction of the embankment to relieve the hydrostatic pressure caused by levee seepage. These relief wells intercept and divert a portion of the groundwater seepage away from the main cooling reservoir. Water loss from the main cooling reservoir results in a buildup of total dissolved solids within the reservoir. The RMPF, located on the west bank of the Colorado River, withdraws makeup water from the river. The main cooling reservoir has a seven-port discharge facility that operates using variable discharge rates ranging from 2.3 to 8.7 cubic meters per second (m^3/s) (80 to 308 cubic feet per second [cfs]) (see ER Section 3.4.2.2). Each port is equipped with a gated valve. A buried pipe, approximately 1.8 km (1.1 mi) in length, conveys water from the reservoir to the discharge ports installed in the Colorado River. The main cooling reservoir also has a spillway near its southeast corner that allows the release of excess water from the main cooling reservoir into the Colorado River during heavy precipitation events. The spillway contains gates that can be manually opened to release water to the Colorado River through a 1,591-m (5,220-ft) -long channel. According to the FSAR of Units 1 and 2 (Version 13, Subsection 2.4.8.2), the spillway capacity is about 86.4 m^3/s (3,050 cfs) at the main cooling reservoir water level of 15.2 m (50 ft) MSL. Both the discharge and spillway facilities are non-safety-related structures, and the addition of STP Units 3 and 4 will have no effect on the operating rules or design of the facilities.

The main cooling reservoir will be part of the closed-loop cooling system of STP Units 3 and 4 during normal operations. To support the operation of STP Units 3 and 4, the applicant will raise the main cooling reservoir normal maximum water surface elevation to 14.9 m (49 ft) MSL.

Groundwater

Section 2.4S.12 of this report describes the staff's review of groundwater characteristics, groundwater users, groundwater well locations, and withdrawal rates.

2.4S.1.5 Post Combined License Activities

There are no post COL activities related to this section.

2.4S.1.6 Conclusion

The staff performed an independent review of the applicant's information in FSAR Section 2.4S.1. The applicant presents and substantiates information relative to the hydrologic description in the vicinity of the site and site regions important to the design and siting of this plant. The staff's review found that the applicant has considered the appropriate site phenomena for establishing the design bases for SSCs important to safety and no outstanding information is required to be addressed in this section. The staff accepted the applicant's approaches used to describe the hydrologic phenomena in the vicinity of the site and site regions.

Accordingly, the staff concluded that the identification and consideration of the safety-related hydrology in the vicinity of the site and site regions are acceptable and meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c). The information addressing the COL Information Item 2.13 is acceptable.

2.4S.1.7 References

Gibeaut, J.C., White, W.A., Hepner, T., Gutierrez, R., Tremblay, T.A., Smyth, R., and Andrews, J., "Texas Shoreline Change Project – Gulf of Mexico Shoreline Change from the Brazos River to Pass Cavallo, Bureau of Economic Geology," The University of Texas, Austin, 2000.

Lower Colorado Regional Water Planning Group, "*2006 Region 'K' Water Plan for the Lower Colorado Regional Water Planning Group*," January 2006; http://www.twdb.state.tx.us/rwpg/2006_RWP/RegionK/.

Lower Colorado River Authority, "*LCRA Dams Form the Highland Lakes,"* 2009; http://www.lcra.org/water/dams/index.html ; accessed March 30, 2009.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Final Safety Analysis Report, Revision 0, Part 2, 2007.

Texas Department of Transportation, "Gulf Intracoastal Waterway, Legislative Report to the 80th Texas Legislature," Austin, Texas, 2007.

2.4S.2 Floods

2.4S.2.1 Introduction

This section of the FSAR discusses the historical flooding at the proposed site or in the region of the site, and summarizes and identifies the individual types of flood-producing phenomena and combinations of flood-producing phenomena considered in establishing the flood design bases for safety-related plant features. This section also covers the potential effects of local intense precipitation.

This SER section provides a review of the following specific areas: (1) a description of the flood history, (2) flood design considerations, and (3) the effects of local intense precipitation.

2.4S.2.2 Summary of Application

In Section 2.4S.2, the applicant addresses the information related to site and regional flood causal mechanisms. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.14 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, flood design considerations, and the effects of local intense precipitation.

2.4S.2.3 Regulatory Basis

The relevant requirements of the Commission regulations for floods, and the associated acceptance criteria, are in Section 2.4.2 of NUREG–0800.

The applicable regulatory requirements for identifying floods are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices

- RG 1.102, “Flood Protection for Nuclear Power Plants.”

2.4S.2.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.2 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to site floods. The staff’s technical review of this application included an independent review of the applicant’s information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff’s evaluation of the technical information in FSAR Section 2.4S.2.

COL License Information Item

- COL License Information Item 2.14 Floods

The staff reviewed site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, flood design considerations, and the effects of local intense precipitation.

2.4S.2.4.1 Flood History

This section describes the historical floods at and in the vicinity of the proposed site.

Information Submitted by Applicant

Flooding near the STP site from natural events includes flooding in the Colorado River, hurricane-induced storm surges, dam and levee breaches, tsunamis, and local flooding in the LRS.

The applicant states in the FSAR Section 2.4S.2 that there are no records of stream flow or stage for the LRS. Using a local probable maximum precipitation (PMP) event, the applicant estimates the local floods that could potentially pose a hazard to safety-related SSCs of STP Units 3 and 4.

The USGS maintains and operates a network of stream gauges at the Colorado River near the vicinity of the STP site. The three gauges closest to the STP site are at Bay City (USGS gauge number 08162500), Wharton (USGS gauge number 08162000), and Columbus (USGS gauge number 08161000). The Bay City and Wharton gauges are more representative of the stream-flow conditions near the STP site because floodplain characteristics upstream of Columbus are different from those near the STP site. The Bay City and Wharton gauges are located approximately 50 to 80.5 km (16 and 50 mi) upstream of the STP site, respectively.

The applicant presents the annual peak stream-flow data at Bay City (for water years 1940 and 1948 through 2006) and at Wharton (water years 1919 through 2006) in FSAR Tables 2.4S.2-1 and 2.4S.2-2, respectively. Flood discharges at these gauges are affected by regulation from several upstream dams. Lake Travis, which was impounded by the construction of Mansfield Dam in 1942, is the largest impoundment in the Colorado River Basin. The highest observed peak discharges at the Bay City and Wharton gauges since the construction of Mansfield Dam are 2381.4 m^3/s (84,100 cfs) on June 26, 1960, and 2118.1 m^3/s (74,800 cfs) on October 23, 1960[1] respectively. The historical peak discharge at the Wharton gauge before the construction of Mansfield Dam is 4502.4 m^3/s (159,000 cfs) on June 20, 1935. The highest recorded flood elevations at the Bay City gauge are 17.1, 16.9 and 16.8 m (56.1, 55.4, and 55.0 ft) MSL in 1913, 1922, and 1929, respectively, before the construction of Mansfield Dam. After the construction of Mansfield Dam, the highest flood elevations at the Bay City gauge were 14.1 and 11.8 m (46.4 and 38.67 ft) MSL in 1960 and 1995 water years, respectively.[2]

During the study of the Colorado River Flood Damage Evaluation Project of the USACE and the LCRA in early 1990s, Halff Associates, Inc. (1992) estimated a flood elevation of 6.4 m (21.0 ft) MSL corresponding to the 2316.3 m^3/s (81,800 cfs) discharge on October 24, 1998, at the Farm-to-Market (FM) 21 Bridge crossing.

At a recently established USGS stream-flow gauge on the Colorado River Bypass Channel near Matagorda (USGS gauge number 08162506), the maximum recorded water surface elevation in the East Colorado River for the period of October 1999 to May 2007 was 2.1 m (7.05 ft) MSL, with a corresponding stream-flow discharge of 2064.3 m^3/s (72,900 cfs) at the Bay City gauge.

FSAR 2.4S.2 states that there are no reported events of ice sheet formation or ice jams for the Colorado River at the STP site or the LRS.

The estimated flood levels from the postulated breach of the main cooling reservoir are higher than the site grade. As a result, the applicant has identified a departure, STP DEP T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's data in FSAR Section 2.4S.2 regarding historical flooding. The staff independently obtained annual peak flow data for the Wharton and Bay City USGS stream-flow gauges. The staff plotted the historical peak flow data for the two gauges in Figures 2.4S.2-1 and 2.4S.2-2.

Based on these data, the staff determined that the historical maximum peak discharges at the Wharton and Bay City USGS gauges are 4,502.4 m^3/s (159,000 cfs) on June 30, 1935 and 2,381.4 m^3/s (84,100 cfs) on June 26, 1980, respectively. Mansfield Dam was constructed in 1942. Before 1942, the peak discharges at the Wharton USGS gauge have shown higher values ranging from 356.8 to 4,502.4 m^3/s (12,600 to 159,000 cfs), with a mean of 1,757.5 m^3/s (62,067 cfs). Since 1942, the peak discharges have ranged from 108.2 to 2,118.1 m^3/s (3,820 to 74,800 cfs), with a mean of 829.9 m^3/s (29,309 cfs). These discharge value estimates are based on recorded stages at each gauge station. The stages before 1942 ranged from 3.5 to

[1] FSAR Section 2.4S.2 has a typographical error. The correct date for the peak discharge of 74,800 cfs at the Wharton gauge is October 23, 1998.

[2] There are other water years in which floodwater elevation has exceeded 39 ft above MSL at the Bay City gauge. These are described in the staff's technical evaluation.

15.8 m (11.5 to 51.9 ft) MSL. The stages after 1942 ranged from 1.7 to 14.8 m (5.7 to 48.7 ft) MSL.

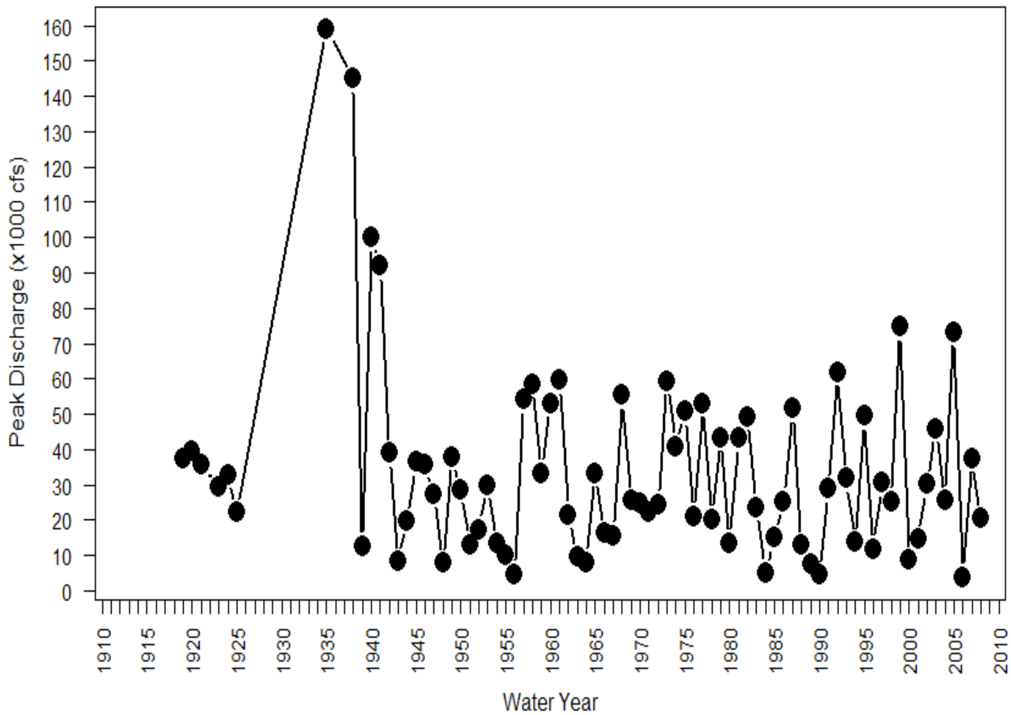

Figure 2.4S.2-1. Peak Stream-Flow Discharge in the Colorado River at the Wharton USGS Gauge

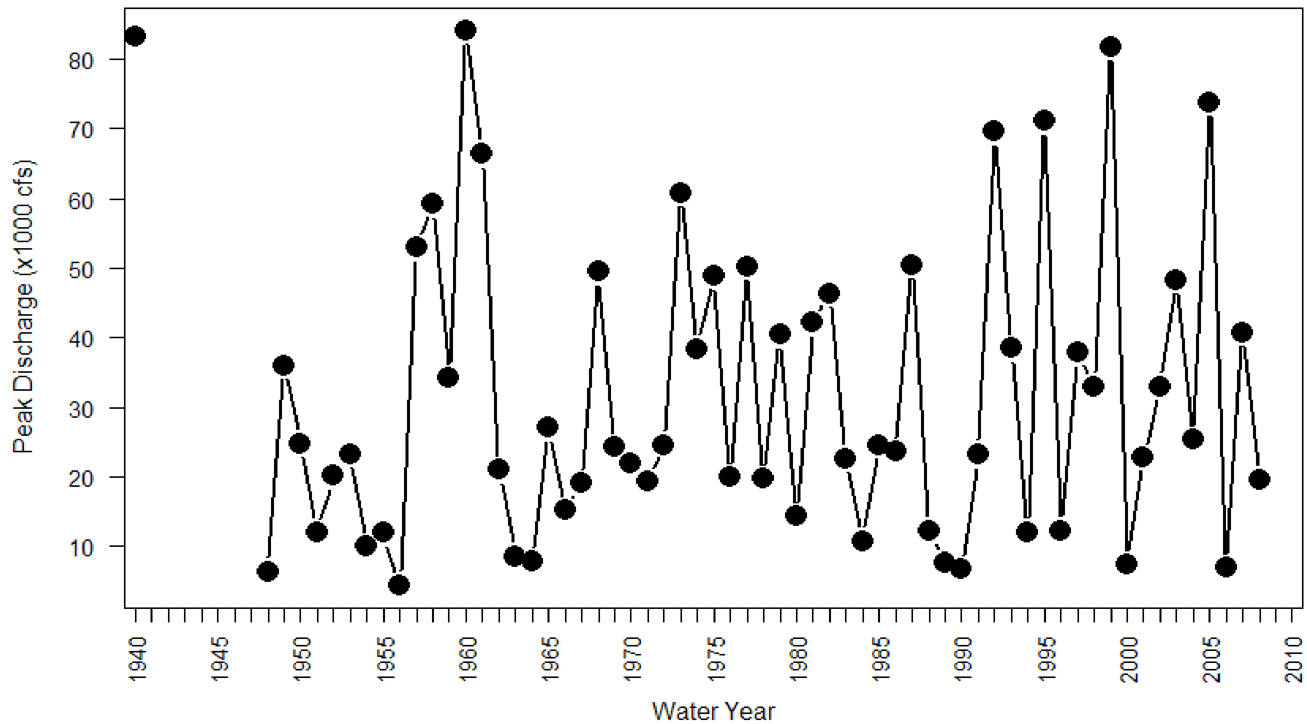

Figure 2.4S.2-2. Peak Stream-Flow Discharge in the Colorado River at the Bay City USGS Gauge

Before 1942, recorded discharges at the Bay City USGS gauge are fully available only in 1940. The annual peak discharge was 2,358.8 m^3/s (83,300 cfs) on July 4, 1940, and the corresponding water level was 14.2 m (46.6 ft) MSL. However, the gauge height during peak stream-flow discharges at the Bay City gauge show consistently higher values ranging from 14.2 to 17.1 m (46.6 to 56.1 ft) MSL. After 1942, the gauge heights during peak stream-flow discharge ranged from 3.8 to 14.1 m (12.5 to 46.4 ft) MSL. Table 2.4S.2-1 shows the maximum gauge heights recorded since 1942.

Table 2.4S.2-1. Ten Highest Water Levels Recorded at the Bay City USGS Gauge Since Construction of the Mansfield Dam in 1942

Date (Water Year)	Peak Discharge (m^3/s) / (cfs)	Water Level (m / ft MSL)
06/26/1960 (1960)	2,381.4 / 84,100	14.1 / 46.4
09/15/1961 (1961)	1,880.2 / 66,400	13.4 / 44.1
10/17/1957 (1958)	1,676.4 / 59,200	13.0 / 42.8
05/01/1957 (1957)	1,500.8 / 53,000	12.7 / 41.8
11/27/2004 (2005)	2,089.8 / 73,800	12.7 / 41.7
10/24/1998 (1999)	2,316.3 / 81,800	12.5 / 41.0
12/27/1991 (1992)	1,970.9 / 69,600	11.9 / 38.9
06/15/1973 (1973)	1,721.7 / 60,800	11.8 / 38.7
10/20/1994 (1995)	2,013.3 / 71,100	11.8 / 38.7
06/26/1968 (1968)	1,401.7 / 49,500	11.4 / 37.5
m=meter; ft=foot; cfs=cubic foot per second; s=second; MSL=mean sea level		

2.4S.2.4.2 Flood Design Considerations

This section describes the scenarios used to determine the design-basis flood at the STP site.

Information Submitted by Applicant

The applicant determines the design-basis flood elevation at the STP site from several scenarios, including the probable maximum flood (PMF) of streams and rivers, potential dam failures, probable maximum surge and seiche flooding, probable maximum tsunamis, flooding due to ice effects, and the potential for flooding caused by channel diversions. The respective FSAR sections describe these flood scenarios. The applicant considers combinations of appropriate conditions with flood scenarios such as wind-generated waves and tidal levels as recommended by the American National Standards Institute (ANSI)/American Nuclear Society (ANS)-2.8–1992 (ANS, 1992).

The applicant estimates the design-basis floodwater surface elevation at the STP site from the postulated breach of the main cooling reservoir embankment. The design-basis flood elevation of 12.2 m (40 ft) MSL is above the site grade and the ground-floor elevation of safety-related SSCs for STP Units 3 and 4. Therefore, all STP Units 3 and 4 SSCs in the power block area below the elevation of 12.2 m (40 ft) MSL will require appropriate flood-protection measures, such as watertight doors and components that will prevent any floodwater intrusion into safety-related areas of the plant. The UHS and the RSW pump house are located at the UHS tower basin and are watertight below the floor slab elevation at 15.2 m (50 ft) MSL. Therefore, they will not need flood protection. FSAR Section 2.4S.10 discusses flood-protection requirements.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A of 10 CFR Part 52. Due to flood levels from the postulated breach of the main cooling reservoir at higher than the site grade, the applicant identifies a departure, STP DEP

T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's description of flooding mechanisms in FSAR Section 2.4S.2 and compared them to the applicable guidance in NUREG-0800, Section 2.4.2. The staff determined that the applicant has considered all plausible flooding mechanisms at the STP site. The corresponding sections of this SER describe the staff's review of the individual flooding mechanisms and their flooding potential. After reviewing the applicant's submittals and the staff's independent confirmatory analyses in Sections 2.4S.2, 2.4S.3, 2.4S.4, 2.4S.5, 2.4S.6, and 2.4S.10 of this SER, the staff determined that the maximum floodwater surface elevation at the STP Units 3 and 4 site would be caused by a postulated failure of the northern main cooling reservoir embankment. The staff confirmed in Section 2.4S.4 of this SER that the design-basis maximum water surface elevation at the STP Units 3 and 4 site is 12.2 m (40 ft) MSL.

2.4S.2.4.3 Effects of Local Intense Precipitation

This section describes the estimation of local intense precipitation and its effects on the safety-related SSCs of STP Units 3 and 4.

Information Submitted by Applicant

Probable Maximum Precipitation Depths

The applicant estimates the design basis for local intense precipitation, which is the all-season, 2.60-km^2 (1-mi^2) PMP from the U.S. National Weather Service (NWS) Hydrometeorological Reports (HMRs) No. 51 and 52 (Schreiner and Riedel 1978; Hansen et al., 1982). FSAR Table 2.4S.2-4 lists the values of the PMP depths, which are reproduced below in Table 2.4S.2-2.

The 1-hour and 5-minute local PMP depths of 50.3 cm (19.8 in.) and 16.3 cm (6.4 in.), respectively, exceed the corresponding ABWR DCD values of 49.3 and 15.7 cm (19.4 and 6.2 in.), respectively. The applicant identifies this exceedance as a departure, STP DEP T1 5.0-1, from the certified design. Justification for the departure is discussed in FSAR Table 2.0-2. Standard ABWR seismic Category I structures are designed with roofs without parapets or with parapets and scuppers that supplement roof drainage to minimize the accumulation of precipitation on the roofs. Site-specific seismic Category I structures, such as RSW pump houses, are designed without parapets to minimize the ponding of water. Therefore, the applicant argues that an exceedance of 1 cm per hour (cm/hr) (0.4 in./hr) in precipitation rate will not result in a substantial increase in roof design load and therefore, will not affect the design of these structures.

Table 2.4S.2-2. Local Intense Precipitation at the STP Site (Adapted from FSAR Table 2.4S.2-4)

PMP Duration and Area	6-hr, 10-mi^2 Ratio	1-hr, Point Ratio	Source	PMP Depth (cm) / (in.)
72 hr, 10 mi^2	–	–	HMR 51 - Fig. 22	141.5 / 55.7
48 hr, 10 mi^2	–	–	HMR 51 - Fig. 21	131.6 / 51.8
24 hr, 10 mi^2	–	–	HMR 51 - Fig. 20	119.6 / 47.1
12 hr, 10 mi^2	–	–	HMR 51 - Fig. 19	96.0 / 37.8
6 hr, 10 mi^2	–	–	HMR 51 - Fig. 18	81.3 / 32.0
3 hr	–	–	Fitted from FSAR Figure 2.4S.2-3	75.4 / 29.7
2 hr	–	–	Fitted from FSAR Figure 2.4S.2-3	67.6 / 26.6
1 hr, point	0.62	–	HMR 52 - Fig. 23	50.3 / 19.8
30 min, point	–	0.73	HMR 52 - Fig. 38	36.8 / 14.5
15 min, point	–	0.50	HMR 52 - Fig. 37	25.1 / 9.9
5 min, point	–	0.32	HMR 52 - Fig. 36	16.3 / 6.4

<u>Local Drainage Components and Subbasins</u>

The site grade in the STP Units 3 and 4 power block area will range from 11.1 m (36.6 ft) MSL in the center to 9.8 m (32 ft) MSL at the corner, with an approximate gradient of 0.4 percent toward the corners. The power block and the UHS will be located inside the security perimeter. Local East and West Channels will collect runoff within the security perimeter and will discharge to the north across through narrow grated openings in concrete security barriers and underground culverts across security fences. These channels join the Main Drainage Channel (MDC) that runs from east to west north of the STP Units 3 and 4 site.

Catch basins will collect runoff from the STP Unit 3 power block area and direct the discharge to the East Channel by connecting drainage pipes. Similarly, runoff from the STP Unit 4 power block area will flow to the West Channel, which also collects runoff from the UHS area. Runoff from the switchyard of STP Units 1 and 2 will flow to the MDC, which also collects runoff from an area bounded by FM 521 to the north.

The MDC flows west parallel to the security barriers north of STP Units 3 and 4, then turns southwest near the northwest corner of the security barrier, and continues flowing southwest before joining the LRS. A little upstream of the west access road, the MDC and LRS are connected by a link channel. At approximately 152 m (500 ft) south of the link channel, both the MDC and LRS cross the west access road via separate culverts.

Using USGS topographic maps, aerial surveys, and locations of roads and barriers, the applicant divides the site drainage area into seven subbasins: North1 (3.797 km^2 [1.466 mi^2]), North2 (0.772 km^2 [0.298 mi^2]), North3 (0.458 km^2 [0.177 mi^2]), PBN1 (0.826 km^2 [0.319 mi^2]), PBW1 (0.127 km^2 [0.049 mi^2]), PBW (0.350 km^2 [0.135 mi^2]), and PBE (0.231 km^2 [0.089 mi^2]).

Peak Discharges

The applicant used the USACE Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) computer model to determine peak discharges for the seven subbasins. The applicant assumed that the whole site drainage is impervious at the start of and during the local PMP event. The applicant estimated the times of concentration for the subbasins using the U.S. Natural Resources Conservation Service (NRCS) recommendations (NRCS, 1986). To account for nonlinear effects during extreme floods, the estimated times of concentration were reduced by 25 percent, as recommended in USACE Engineering Manual EM-1110-2-1417 (USACE, 1994). The applicant estimated the lag time as 60 percent of the corresponding time of concentration described by the USACE (2006).

The LRS passes under FM 521 through pipe culverts. The applicant assumed that during the local PMP event, runoff upstream of FM 521 will accumulate while there will be some runoff contribution to LRS via the pipe culverts. After the runoff accumulation results in overtopping FM 521, more runoff from the north of FM 521 will contribute to the LRS.

The applicant sets up the site hydrologic model in HEC-HMS as shown in the hydrologic diagram (FSAR Figure 2.4S.2-6) using the subbasin areas, local PMP intensities, lag times, a runoff curve number of 98 representing impervious areas, and the NRCS dimensionless unit hydrograph option. FSAR Table 2.4S.2-2 shows the subbasin properties, peak discharges, and times to peak. The applicant uses the site hydrologic model to compute the runoff hydrograph during the local PMP event. Because of longer lag times, the storage of runoff upstream of FM 521, and the subsequent overtopping of FM 521, the combined peak discharge from subbasins North1 and North2 occurs at hour 6:25 at the upstream boundary of the LRS. Therefore, the peak discharge into the LRS at its confluence with the MDC occurs much later than the flood peak in the MDC, which also receives runoff from subbasins PBN1, PBE, PBW, and PBW1. FSAR Table 2.4S.2-6 shows the peak discharge at various locations within the site drainage area. The applicant estimates that the peak discharge at the outfall where the LRS and the MDC meet is 279 m^3/s (9,852 cfs).

Hydraulic Model Setup

The applicant estimates the maximum water surface elevation during the local site flooding under a local PMP event using the USACE Hydrologic Engineering Center-River Analysis System (HEC-RAS) model (USACE, 2005). The applicant develops the cross sections at several places on the LRS, MDC, and East and West Channels for inclusion in the hydraulic model (FSAR Figure 2.4S.2-7). The applicant obtains the bottom elevations, longitudinal slopes, and side slopes of the channels from site design details (the MDC and East and West Channels) or from an aerial survey (the LRS).

The applicant inputs the inflow discharges in the HEC-RAS model from estimated HEC-HMS discharge hydrographs. In the HEC-HMS computations, peak discharge at the outflow of the site area occurs within 25 minutes of the peak discharges for subbasins PBE, PBW, PBW1, and PBN1. Therefore, the applicant conservatively assumes that peak discharge in each of these individual subbasins coincides with the peak discharge at the outlet (hour 3:35), which also makes the peak discharge into the HEC-RAS model greater than that computed by the HEC-HMS model. In contrast, the peak discharge into the LRS occurs much later (hour 6:25). Therefore, the applicant specifies the input discharge to the LRS as the discharge at hour 3:35 from its HEC-HMS discharge hydrograph. The applicant distributes the peak discharge from each subbasin to the corresponding channel reach using a proportioning approach. The peak discharge for the most upstream cross section in a channel reach is proportional to the contributing area upstream of that reach. The applicant obtains the peak discharges for the remaining cross sections by subtracting the peak discharge at the most upstream cross section from the peak discharge for the whole basin, and then dividing the remainder by the number of remaining cross sections.

The applicant assumes the pipe culverts through which the MDC and LRS cross the west access road to be completely blocked during the local PMP event. Therefore, the applicant models the west access road as an in-line weir in the HEC-RAS. The applicant estimates the width and breadth of the weir from an aerial survey using a weir coefficient of 2.6. The applicant uses Manning's n values recommended by Chow (1959).

The applicant uses the HEC-RAS model to simulate steady-state, subcritical flow conditions in the site drainage area. A sensitivity analysis of the model indicates that the flow over the weir at the west access road is controlled by upstream boundary conditions if the water surface elevation downstream of the weir is less than 10.4 m (34 ft) MSL. It is unlikely that water surface elevations downstream of the west access road will exceed 10.4 m (34 ft) MSL, because most of the runoff upstream of the weir is intercepted by the west access road. Therefore, the applicant uses a constant water level of 10.4 m (34 ft) MSL as the downstream boundary condition in the HEC-RAS simulation.

Flood Elevations

The applicant estimates the maximum water surface elevation in the power block area to be 11.2 m (36.6 ft) MSL from the HEC-RAS simulation. Because this water surface elevation is less than that from the breach of the main cooling reservoir embankment, flood from a local PMP event on the site does not result in the design-basis flood.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Due to higher-than-site-grade flood levels from the postulated breach of the main cooling reservoir, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. The applicant provides information to address COL License Information Item 2.14 from the generic DCD.

NRC Staff's Technical Evaluation

Probable Maximum Precipitation Depths

The staff reviewed the description of the applicant's local PMP. The staff determined that the applicant's method is acceptable because SRP Section 2.4.2 recommends that method. In an independent analysis, the staff estimated the local PMP from HMR 51 and 52 and obtained values that closely match the applicant's values in FSAR Section 2.4S.2. Therefore, the staff agreed with the applicant's local PMP depth estimates.

FSAR Table 2.0-2 shows that the precipitation site characteristic at the STP site, defined by the local PMP rate, is 50.3 cm/hr (19.8 in./hr), which exceeds the ABWR DCD envelope value of 49.3 cm/hr (19.4 in./hr). The staff issued RAI 02.04.02-1 requesting the applicant to discuss the additional load on safety-related SSCs as a result of this exceedance and to demonstrate that sufficient safety margins exist in the design of these SSCs.

In a letter dated June 12, 2008 (ML081710126), the applicant's response to RAI 02.04.02-1 states that the reactor building, the control building, and two RSW pump houses are the only safety-related SSCs that will be affected by the local PMP. The applicant also states in FSAR Tier 2, Subsections 3H.1.4.2 and 3H.2.4.2.5, that the roofs of the safety-related SSCs are either designed without parapets or with scuppers. The applicant adds that these roof designs meet the provisions of RG 1.102.

The staff reviewed the applicant's response and determined that the safety-related SSCs for Units 3 and 4 will be designed so that either their roofs have no parapets or the roofs are equipped with scuppers. The staff determined that the slight exceedance of 2.1 percent in the design-basis roof load due to the local PMP site characteristic would not result in excessive ponding, because the scuppers would assist in draining ponded water away from the roofs of safety-related SSCs. Therefore, RAI 02.04.02-1 is closed.

Local Drainage Components and Subbasins

The staff reviewed the description of site drainage components and subbasins the applicant includes in FSAR Section 2.4S.2. The staff determined that this description matches the staff's observations of the site during safety and environmental site visits. The staff agreed, therefore, with the applicant's description of local drainage components and subbasins.

Peak Discharges

The applicant selected the USACE HEC-HMS model to estimate peak discharges in the site drainage area under a local PMP event. The staff agreed that HEC-HMS is an appropriate computer model to apply when determining the peak discharge from local site drainages. This model is one of the USACE models recommended in SRP Sections 2.4.3 and 2.4.4.

The applicant provided the HEC-HMS input and output files in electronic format. The staff reviewed the applicant's modeling work and determined that these data are sufficient to adequately estimate peak discharges.

The staff issued RAI 02.04.02-3 requesting the applicant to discuss (a) flood magnitude and timing; (b) the effect on water levels in the power block area; and (c) the effect of the 10.4-m (34-ft) MSL constant water-level boundary condition in the HEC-RAS simulation, if local access road FM 521 does not act like a barrier and flood runoff from the North1 and North2 subbasins is not significantly lagged. The staff also asked the applicant to (d) justify using a 6-hour PMP rather than a PMP value of a shorter duration and more intensity to obtain peak PMF water surface elevations in the power block area; and (e) specify in the FSAR the point where the peak floodwater surface elevation is simulated within the power block area. The applicant's responses to subparts (a) and (d) are relevant to the discussion in this subsection. The responses to the other subparts are described in the "Flood Elevations" subsection below.

In a letter dated August 12, 2008 (ML091811141), the applicant's response to RAI 02.04.02-3 refers to the local PMF analysis in the FSAR as the COL application base case. The applicant provides two modeling scenarios with respect to how FM 521 affects the peak discharges near the power block area. In the first scenario, the applicant assumes that FM 521 will not act as a barrier to runoff generated in the North1 and North2 subbasins, and the combined runoff will discharge at the top of the LRS reach. In the second scenario, the applicant assumes that FM 521 will not act as a flow barrier at all. The applicant therefore concludes that the north subbasin of the local site drainage area will consist of a single, larger subbasin that will include the drainage areas of North1, 2, and 3 subbasins. This single, larger north subbasin will discharge directly at the outfall location (the junction where the LRS and the MDC meet).

The applicant states in the response to RAI 02.04.02-3 that the first of the two scenarios resulted in a higher peak discharge at the bottom of the LRS (approximately 275.1 m^3/s [9,715 cfs]) than the COL application base case (approximately 217.7 m^3/s [7,687 cfs]). The peak discharge also occurred earlier in the first scenario—5 hours and 25 minutes after the beginning of the local PMP storm—compared to the COL application base case timing of 6 hours and 25 minutes after the beginning of the local PMP storm. The applicant also reports that the predicted peak discharge at the outlet for the first scenario was approximately 324.5 m^3/s (11,460 cfs), which is greater than the 279 m^3/s (9,852 cfs) for the COL application base case and occurs at nearly the same time (3 hours and 35 minutes compared to 3 hours and 40 minutes, respectively, after the beginning of the local PMP storm).

The staff reviewed the applicant's responses and found that the applicant's modeled scenarios represent a reasonable sensitivity analysis for the COL application base case results in the FSAR. The staff independently performed the HEC-HMS simulations using the applicant's input files and confirmed that the applicant's reported simulated peak discharges are accurate. The applicant states that the local intense precipitation data used to estimate discharges near the power block area consist of several shorter duration rainfall depths corresponding to 5 minutes, 15 minutes, 1 hour, 2 hours, 3 hours, and 6 hours. The applicant therefore concludes that the effects of more intense precipitation corresponding to durations shorter than 6 hours are captured by the local PMP distribution.

The staff reviewed the applicant's response regarding the precipitation distribution used to estimate flood discharges during the local intense precipitation event. The staff agreed with the applicant's statement that the higher expected intensity of precipitation for events with a shorter duration is represented within the distribution used by the applicant.

Hydraulic Model Setup

The applicant selects the USACE HEC-RAS model to simulate the hydraulics of flooding in site drainage channels and the adjacent LRS during the local PMP event. Because this model is one of the recommended models in the SRP, the staff determined that the HEC-RAS is an appropriate model to apply to the simulation of channel hydraulics during the local PMP event.

The staff reviewed the applicant's modeling work and determined that the data were sufficient for the staff's review and subsequent confirmatory analysis. The staff used the applicant's model to carry out an independent confirmatory analysis of the peak floodwater elevations in the site drainage area under the local PMP event.

Flood Elevations

The applicant uses the USACE HEC-RAS model to estimate flood elevations at the site during the local PMP event. The staff determined that the HEC-RAS is an appropriate model for this purpose, because this model was supported and widely used by the U.S. Army Corps of Engineers.

The staff issued RAI 02.04.02-2 requesting the applicant to provide input and output files used in the HEC-RAS simulations. In attachments to a letter dated June 12, 2008 (ML081710126), the applicant provided the HEC-RAS input and output files in electronic format.

The applicant responded to RAI 02.04.02-3 in a letter dated August 12, 2008 (ML091811141). The applicant’s responses to subparts (b), (c), and (e) are relevant to the discussions in this subsection.

The applicant states that steady flow routing in the HEC-RAS was used to estimate water surface elevations near the power block area. The applicant specified inflows into the HEC-RAS model cross sections using the time when the peak discharge occurred at the outfall—3 hours and 40 minutes after the beginning of the storm for the reaches, the LRS, and North3—which had a peak discharge time significantly different from 3 hours and 40 minutes. The applicant used peak discharges for the other reaches regardless of their timing. The applicant notes that this approach is similar to that used in the COL application base case. The resulting peak discharge at the outfall was about 376 m^3/s (13,293 cfs), or approximately 20 percent higher than the 313.8 m^3/s (11,080 cfs) used in the COL application base case.

The applicant states that for the first scenario simulation, the maximum water surface elevation near the power block area was 11.22 m (36.8 ft) MSL. The maximum water surface elevation occurred in the East Channel at three locations: the most upstream river station and two cross sections near the proposed location of the Unit 3 reactor building. The maximum simulated water surface elevation is slightly higher than the 11.16 m (36.6 ft) MSL in the COL application base-case simulation. The applicant also states that the higher water surface elevation is a result of the conservative assumption related to the effect of the postulated FM 521 breach, which ignores any attenuation of flood peaks due to backwater effects at the breach. The assumption also ignores any diversion of flood flow away from the LRS and MDC following the FM 521 breach.

The applicant states that the peak elevation of the floodwater surface of 11.16 m (36.6 ft) MSL will occur in the East Channel within the protected area boundary and will affect the safety-related reactor and control buildings. The applicant also states that the peak elevation of the floodwater surface along the entire West Channel will be about 11.1 m (36.4 ft) MSL. Therefore, the applicant conservatively assumes that the entire power block area will be affected by a maximum elevation of the floodwater surface of 11.16 m (36.6 ft) MSL as a result of local intense precipitation. The applicant has updated the FSAR to state the maximum water surface elevation within the power block area of STP Units 3 and 4.

The staff reviewed the applicant’s response and agreed that the model represents a conservative scenario in terms of flooding near the power block area. Based on the minor increase in the maximum elevation of the floodwater surface under conservative assumptions regarding the FM 521 breach, the staff determined that the maximum elevation of the floodwater surface near the power block area would be less than the design-basis elevation of the floodwater surface resulting from the main cooling reservoir breach. The staff concluded that flooding near the power block area resulting from the local intense precipitation event is not the controlling flood scenario at the STP Units 3 and 4 site. Therefore, RAI 02.04.02-2 and RAI 02.04.02-3 are closed. In response to RAI 02.04.02-3, the applicant proposed to revise the first paragraph of FSAR Section 2.4S.2.3.5 to specify the spot at which the peak flooding level was simulated. FSAR, Revision 6, reflects these changes and Confirmatory Item 02.04.02-1 is therefore closed.

The staff needed more detailed information on the HEC-RAS model to understand the procedure used to evaluate the model's conservatism. The staff issued RAI 02.04.02-4 requesting the applicant to elaborate on the following statements in FSAR Subsection 2.4S.2.3.4, page 2.4S.2-8: "The peak discharge obtained for a subbasin in HEC-HMS was first distributed to the most upstream cross section of a stream reach in the HEC-RAS in proportion to the area contributing to that cross section and the total area of the subbasin. The remaining portion of the peak discharge is then distributed equally among the remaining cross sections within the receiving channel reach."

In a letter dated July 9, 2008 (ML081960070), the applicant's response to RAI 02.04.02-4 states that the discharges simulated by the HEC-HMS for each of the subbasins that drain into the HEC-RAS channel reaches were distributed among the cross sections within the reach based on the drainage area upstream of the respective cross section. The applicant also states that for the North3 subbasin, which drains into the LRS, the selected flood flow from North3 at the time of peak discharge at the outlet was divided among the 11 cross sections of the LRS. The applicant specifies the discharge for the most upstream cross section of the LRS as the inflow from the storage element at its upstream end, which receives inflows from the North1 and North2 subbasins plus one-eleventh of the flood discharge from the North3 subbasin. The applicant notes that each downstream cross section of the HEC-RAS LRS reach receives an additional one-eleventh of the flood discharge from North3. The applicant uses a similar approach to distribute the flood discharge from subbasin PBN1. The applicant states that approximately 0.26 km^2 (0.1 mi^2) of the PBN1 drainage area directly discharges into the most upstream cross section of the HEC-RAS MDC reach. To estimate this discharge, the applicant multiplies the peak discharge from PBN1 by the ratio of 0.26 km^2 (0.1 mi^2) to the total drainage area of PBN1, 0.83 km^2 (0.319 mi^2). The applicant distributes the rest of the peak discharge from PBN1 among the 27 cross sections of the MDC reach.

The staff reviewed the applicant's response and concluded that it is a reasonable approach for specifying discharges from adjacent drainage areas into each of the HEC-RAS cross sections used in the simulation of the elevation of the floodwater surface. The staff thus determined that the applicant's response is satisfactory. Therefore, RAI 02.04.02-4 is closed.

2.4S.2.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.2.6 Conclusion

The staff reviewed the applicant's submittals in FSAR Section 2.4S.2 and in response to the RAIs. Based on this review, the staff determined that the applicant has appropriately described the flood history, flood causal mechanisms, local intense precipitation, and the estimation of the local PMF near the STP site and no outstanding information is expected to be addressed in this section. The staff found that the applicant has considered the appropriate site phenomena for establishing the site flood causal mechanisms. The staff accepted the methodologies used to determine the local intense precipitation, flood causal mechanisms, and controlling flood mechanisms. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

The staff reviewed the applicant's estimated local intense precipitation rates and found that the applicant's estimated values closely match those estimated independently by the staff. The staff also found, based on independent confirmatory analyses, that the applicant had used a conservative approach to estimate the flood levels at and near the power block area of proposed STP Units 3 and 4. In conclusion, the applicant has provided sufficient information for satisfying 10 CFR Part 52 and 10 CFR Part 100. The information addressing COL License Information Item 2.14 is adequate and acceptable.

2.4S.2.7 References

American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," *Historical Technical Reference*, American Nuclear Society ANSI/ANS-2.8, July 1992.

Chow, V. T., *Open-Channel Hydraulics*, McGraw-Hill Book Co., New York, NY, 1959.

Halff Associates, Inc., "1992 Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II," prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, July 2002.

Hansen, E.M., L.C. Schreiner, and J.F. Miller, "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 52, National Weather Service, Washington, DC, 1982.

Natural Resources Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release 55, U.S. Department of Agriculture, Natural Resources Conservation Service, 2nd Ed., revised June 1986, update of Appendix A, January 1999.

Pararas-Carayannis, G., "Verification Study of a Bathystrophic Storm Surge Model," U.S. Army Corps of Engineers Technical Memorandum No. 50, Coastal Engineering Research Center, Fort Belvoir, VA, May 1975.

Schreiner L.C., and J.T. Riedel, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," NOAA Hydrometeorological Report 51, National Weather Service, Washington, DC, 1978.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

U.S. Army Corps of Engineers, "Flood-Runoff Analysis," U.S. Army Corps of Engineers Engineer Manual 1110-2-1417, Washington, DC, August 1994.

U.S. Army Corps of Engineers, "HEC-RAS, River Analysis System, User's Manual," Version 3.1.3, Hydrologic Engineering Center, 2005.

U.S. Army Corps of Engineers, "Hydrologic Modeling System, HEC-HMS," User's Manual, Version 3.1.0, U.S. Army Corps of Engineers, Hydrologic Engineering Center, Washington, DC, November 2006.

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

2.4S.3.1 Introduction

This section of the FSAR describes the hydrological site characteristics affecting any potential hazard to the plant's safety-related facilities as a result of the effect of the PMF on streams and rivers.

Section 2.4S.3 of this SER provides a review of the following specific areas: (1) regional probable maximum precipitation and precipitation losses, (2) runoff and stream course models, (3) PMF, (4) consideration of other site-related evaluation criteria, and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.3.2 Summary of Application

In Section 2.4S.3, the applicant addresses the information about site-specific PMFs on streams and rivers. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.15 identified in DCD Tier, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.15 Probable Maximum Flood on Streams and Rivers

COL License Information Item 2.15 requires COL applicants to provide the basis for determining the protection of safety-related structures against a PMF.

2.4S.3.3 Regulatory Basis

The relevant requirements of the Commission regulations for identifying the PMF on streams and rivers, and the associated acceptance criteria, are in Section 2.4.3 of NUREG-0800.

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices

2.4S.3.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.3 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to the PMF. The staff’s technical review of this application included an independent review of the applicant’s information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff’s evaluation of the technical information in FSAR Section 2.4S.2.

COL License Information Item

- COL License Information Item 2.15 Probable Maximum Flood on Streams and Rivers

The staff reviewed site-specific information related to a PMF and the potential for flooding at the plant site, including the effects of local intense precipitation.

2.4S.3.4.1 Probable Maximum Precipitation

Information Submitted by the Applicant

The applicant estimates the PMP over the Colorado River Basin below the Mansfield Dam in FSAR Section 2.4S.3.1. The applicant’s analysis is based on the PMP established in several studies, namely:

- Updated Final Safety Analysis Report (UFSAR) for STP Units 1 and 2 (STPEGS, 2006)

- A PMF analysis conducted for Mansfield Dam (USBR, 1985, 1989, 2003, 2007; Goodson and Associates, 1990)

- A dam safety analysis for the Lower Colorado River (Freese and Nichols, 1992)

- A flood damage study for the Lower Colorado River (Halff Associates, Inc., 2002).

The applicant follows the procedures described in National Oceanic and Atmospheric Administration (NOAA) NWS HMRs 51 and 52 (Schreiner and Riedel, 1978; Hansen et al., 1982) to obtain the spatial distribution of the PMP within the basin. The applicant estimates the critical centering of the PMP storm pattern that would produce the greatest volume of precipitation within the drainage basin. The applicant analyzes two different storm pattern orientations for the drainage basin to derive the most critical PMF hydrographs near the STP Units 3 and 4 site.

Previous studies (Halff Associates, Inc., 2002 and STPEGS, 2006) used a 96-hour PMP storm duration because the peak discharge from the Upper Colorado River Basin reaches Mansfield Dam and the peak discharge from areas in the Lower Colorado River Basin reaches Wharton by the end of the storm event. The applicant also selects the 96-hour duration storm as the PMP hyetograph for estimating the PMF at the STP Units 3 and 4 site.

The applicant notes that previous studies (USBR, 1985) demonstrate that the largest floods in the Colorado River Basin result from frequent and intense summer rainfall events. Therefore, the applicant does not consider snowmelt or rainfall on antecedent snowpack in estimating the PMF in the Lower Colorado River Basin.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.3 (STPNOC, 2007). The applicant uses NOAA NWS HMR 51 and 52 to estimate the PMP in the Lower Colorado River Basin. The staff verified the 6-, 12-, 24-, 48-, and 72-hour PMP depths from HMR 51 for the subbasin CC-06 which was identified previously (Halff Associates, Inc., 2002) as the center of the critical storm that produces the largest flow rate at Bay City. Based on this review, the staff determined that the applicant's estimates are reasonable.

HMR 51 provides PMP depths for durations up to 72 hours only. The staff reviewed the applicant's method for extrapolating the PMP depths for the 96-hour duration. Generally, the rate of increase in the precipitation depth reduces as the duration of precipitation increases. Therefore, the slope of the depth-duration relationship becomes flatter with increasing duration. For the CC-06 subbasin, the incremental PMP depths for the second day (hours 24 to 48) and the third day (hours 72 to 96) are 13.1 and 7.9 cm (5.2 and 3.1 in.), respectively. The applicant's extrapolation to the fourth day (hours 72 to 96) resulted in an additional 6.6 cm (2.6 in.) of precipitation depth. The staff's review determined that the applicant's method for extrapolating the PMP depth is reasonable, because the relationship between the incremental PMP depth and duration shows a persistent decreasing trend.

The applicant uses the HMR 52 spatial distribution of the PMP pattern. HMR 52 recommends PMP estimation for basin areas equal to or less than 51,780 km^2 (20,000 mi^2). The applicant uses two separate storm pattern for the 57,721 km^2 (22,682 mi^2)-Lower Colorado River Basin-one for the upper and the other for the lower part of the basin. HMR 52 recommends using a single PMP storm pattern, but this approach for the 57,721 km^2 (22,682 mi^2)-Lower Colorado River Basin would result in less precipitation compared to the applicant's approach that uses two separate storm patterns—one for the upper and the other for the lower part of the basin, both of which are smaller in area than the whole basin and therefore would result in PMP intensities that are higher than a hypothetical 57,721 km^2 (22,682 mi^2)-PMP. The use of the two storm patterns would result in additional precipitation falling on the remote areas of an elongated basin. Therefore, the staff accepts the applicant's two storm-pattern approach because it is more conservative and would result in a larger flood at the STP Units 3 and 4 site.

Because snow accumulations in the Lower Colorado River Basin occur infrequently, the staff agreed with the applicant that a snowmelt or rain-on-snow event is unlikely to produce a PMF in the Lower Colorado River Basin (see staff's evaluation in Section 2.4S.3.4.3 below).

2.4S.3.4.2 Precipitation Losses

Information Submitted by Applicant

The applicant discusses precipitation losses in FSAR Section 2.4S.3.2 and Subsection 2.4S.3.4.2.1 (STPNOC, 2007). The applicant assumes no initial losses in the HEC-HMS modeling. The applicant uses guidelines from the Federal Energy Regulatory Commission (FERC, 2001) to specify a uniform continuing loss rate of 0.13 cm/hr (0.05 in./hr).

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.03-7(c) requesting the applicant to discuss how the constant precipitation loss rate of 0.13 cm/hr (0.05 in./hr), adopted for the PMF study, is conservative, as the applicant states in FSAR, Subsection 2.4S.3.4.2.1. In a letter dated July 2, 2008 (ML081890239), the applicant's response to RAI 02.04.03-7(c) states that a uniform continuing loss rate of 0.13 cm/hr (0.05 in./hr) was used to estimate the PMF, and that this value is the minimum range recommended by FERC (2001). The applicant has updated the discussion of precipitation losses in the FSAR. The staff reviewed the applicant's reference and agreed that the precipitation loss rate used in the PMF study is conservative. Therefore, RAI 02.04.03-7(c) is closed.

2.4S.3.4.3 Runoff and Stream Course Models

Information Submitted by Applicant

The applicant discusses the runoff model in FSAR Section 2.4S.3.3 (STPNOC, 2007). Halff Associates, Inc. (2002) developed the HEC-HMS model that included 80 subbasins in the Lower Colorado River Basin extending from below the Mansfield Dam to Bay City. Halff Associates, Inc., calibrated this model to simulate floods up to 100-year storm events. The applicant modifies the Halff model conservatively by decreasing runoff lag times by 25 percent, as recommended by the USACE (1994), and by using modified rating curves for the channel reaches to account for larger flows during the PMF event.

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.03-7(b) requesting the applicant to provide details about how it reached the following conclusion: "snow melt and antecedent snow pack are not a factor in the production of floods at the STP 3 & 4 site," in FSAR Section 2.4S.3.1. In a letter dated July 2, 2008, the applicant's response to RAI 02.04.03-7(b) (ML081890239) states that previous studies of PMF in the Colorado River Basin have noted that frequent and intense rainfall events occurring simultaneously over several subbasins produced the largest recorded floods in the river. The rainfall distribution during a year in the Colorado River Basin has two peaks, one in May and one in September. Spring rainfall events are produced by convective thunderstorms, while late summer or early fall rainfalls are associated with tropical cyclones. The applicant also states that because the climate in the Colorado River Basin is not suitable for an appreciable snowpack development, snow melt or rainfall on antecedent snowpack will not produce a PMF in the Lower Colorado River Basin near the STP Units 3 and 4 site.

The staff reviewed the applicant's information and determined that the hydrometeorological characteristics of the Colorado River Basin, especially the Lower Colorado River Basin, are not suitable for the development of large snowpacks or winter floods. The staff concluded, therefore, that snow melt and rainfall on antecedent snowpack would not cause a PMF at the site.

The staff reviewed the above runoff and stream course models used by the applicant, and concluded that the applicant has appropriately selected numerical models and has used appropriate data sets and parameter values to represent the hydrologic characteristics of the Lower Colorado River Basin. Therefore, RAI 02.04.03-7(b) is closed.

2.4S.3.4.4 Probable Maximum Flood Flow

Information Submitted by Applicant

The applicant discusses the estimation of PMF flow in FSAR Section 2.4S.3.4 along with details of the previous studies. FSAR Table 2.4S.3-1 summarizes estimates of the peak flow at Mansfield Dam from different studies for comparison (STPNOC, 2007). The applicant bases the PMF scenarios at the STP Units 3 and 4 site on the PMF scenarios considered for STP Units 1 and 2. The applicant eliminated some of the previous scenarios because of abandoned plans to build the Shaw Bend Dam on the Lower Colorado River. The three remaining scenarios are as follows:

- Scenario 1: A PMF for the area between Mansfield Dam and Bay City combined with a 3-day antecedent storm equal to 40 percent of the PMP event occurring over the same area 3 days before the PMF event, plus the Mansfield Dam release and the base flow at Bay City.

- Scenario 2: A PMF for the area above Mansfield Dam resulting from a PMP storm in the drainage area from Lake O.H. Ivie to Mansfield Dam, plus a sequential storm equal to 40 percent of the PMP event occurring over the drainage area between Bay City and Mansfield Dam 3 days after the PMP storm upstream of Mansfield Dam combined with the base flow at Bay City.

- Scenario 3: A PMF for the entire Lower Colorado River Basin area between Lake O.H. Ivie and Bay City, with an antecedent Standard Project Storm for the same area added to the base flow at Bay City (Halff Associates, Inc., 2002).

The applicant uses the scenario that produces the highest PMF discharge as the most critical. Based on the previous studies and additional hydrodynamic modeling analyses, the applicant concludes that Scenario 1 is the critical scenario, and uses it to establish the PMF peak discharge of 39,571 m^3/s (1,397,432 cfs) (FSAR Section 2.4S.3.4.3) (STPNOC, 2007).

NRC Staff's Technical Evaluation

The staff reviewed the applicant's modeling approach for assessing the regional PMF in the Lower Colorado River Basin. The staff found that the applicant's selection of the numerical model and the associated parameters are appropriate and that the basin representation within the model is acceptable. As discussed in the Subsection 2.4S.3.4.3 of this SER, the staff noted that the applicant uses conservative assumptions and input parameters such as rainfall distributions and loss rates. Therefore, the staff concluded that the applicant's estimate of the

PMF discharge into the Colorado River near the STP Units 3 and 4 site is appropriate and conservative.

2.4S.3.4.5 Water Level Determinations

Information Submitted by Applicant

The applicant discusses water-level determinations in FSAR Section 2.4S.3.5 (STPNOC, 2007). To put the estimated flood level in context, the applicant uses the following elevations:

- the site nominal grade for safety-related facilities: 10.4 m (34.0 ft) MSL (FSAR Section 2.4S.4)

- the site safety-related entrance slab elevation: 10.7 m (35.0 ft) MSL (FSAR Section 2.4S.4)

- the referenced ABWR DCD site flood level is 0.3 m (1 ft) below the nominal grade: 10.1 m (33.0 ft) MSL

- all ventilation openings of safety-related buildings are located at or above 12.2 m (40 ft) MSL (FSAR Subsection 2.4S.4.3.2).

The applicant uses the HEC-RAS computer program to determine the flood level at the site corresponding to the PMF peak discharge. The Halff study (Halff Associates, Inc., 2002) developed and calibrated the HEC-RAS steady-state model for simulating the floods in the Lower Colorado River Basin. The applicant uses the same model but changes some parameter values from the Halff study. For example, the applicant increases the Manning’s roughness coefficients by 20 percent from the calibrated values used in the Halff study to account for the increased roughness in the overbank and floodplain areas where the PMF discharge is expected to occur. The applicant states that, because the calibrated Manning’s roughness coefficients cannot be used for a hypothetical high-magnitude flood such as a PMF event, the applicant increases the roughness coefficients based on the published recommendations (Smith, 1992). The applicant also expands the width of the river cross sections to simulate adequately the PMF discharges on the floodplain.

The applicant sets the downstream water surface elevation boundary condition to a normal flow depth. Using the HEC-RAS model with conservatively higher roughness coefficient values of the floodplain than those used in the Halff study, the applicant determines the normal water depth at the downstream boundary to be 5.3 m (17.5 ft) NAVD88. The applicant’s estimates of the normal depth with the Halff study roughness value is 4.9 m (16.2 ft) NAVD88. Using the HEC-RAS model with the above downstream boundary condition and the PMF inflow into the basin, the applicant estimates a water surface elevation at the site of 8.0 m (26.1 ft) NAVD88, which is approximately 2.7 m (9 ft) lower than the STP Units 3 and 4 site grade (FSAR Figure 2.4S.3-2). The applicant states that the above PMF flood-level estimate is higher (thus more conservative) than the one estimated with the Halff study roughness values of the floodplain.

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.03-6 requesting the applicant to explain why it states in FSAR Subsection 2.4S.3.5.3.1 that the water level in the Colorado River at the most downstream cross section used in the HEC-RAS model is unaffected by tidal conditions. The applicant's response dated July 9, 2008 (ML081960070), states that under PMF conditions, the discharge into the Colorado River will be 39,570 m^3/s (1,397,432 cfs) at the downstream boundary, and the corresponding normal depth of flow will be an estimated 5.3 m (17.5 ft) NAVD88 or 5.4 m (17.7 ft) NGVD29. The applicant reports the maximum water level recorded at the NOAA tide gauge at Freeport, Texas, as 1.51 m (4.95 ft) MSL. Because the PMF water surface elevation at the normal depth exceeds the maximum tidal level, the applicant concludes that the normal depth at the downstream boundary is the appropriate boundary condition to use in the HEC-RAS model.

The staff reviewed the applicant's response and agreed that the large PMF discharge would occur at a greater depth of flow at the downstream boundary of the HEC-RAS modeling domain and would therefore be unaffected by tidal conditions.

The staff also reviewed the applicant's approach for estimating the elevations of floodwater surface near the STP Units 3 and 4 site during a PMF in the Lower Colorado River Basin. The staff determined that the applicant has appropriately selected the numerical model, HEC-RAS, and its associated parameter values and boundary conditions. The staff also found that the applicant adopts the conservatively estimated flood discharges obtained from the HEC-HMS model. The staff concluded that the applicant has appropriately and conservatively estimated the PMF water surface elevation near the STP Units 3 and 4 site. Therefore, RAI 02.04.03-6 is closed.

2.4S.3.4.6 Coincident Wind Wave Activity

Information Submitted by Applicant

The applicant discusses the coincident wind-wave activity in FSAR Section 2.4S.3.6. The applicant does not estimate the coincident wind-wave activity with PMF because the flood elevations for the upstream dam failure and the main cooling reservoir embankment breach scenarios are estimated to be much higher than that of the regional PMF. The applicant concludes that PMF water surface elevations combined with wind waves will not be the controlling scenario at the STP Units 3 and 4 site.

NRC Staff's Technical Evaluation

The applicant does not provide estimates of wave heights from wind wave activity for the PMF water surface elevations. The staff agreed with the applicant that any wind-wave activity coincident with the PMF in the Colorado River would be smaller than that estimated for the upstream dam-failure scenario and therefore would not exceed the estimated elevation of the floodwater surface for that scenario. The staff concludes that estimating the wind-wave effects coincident with the PMF in the Colorado River near the STP Units 3 and 4 site is not necessary.

The staff reviewed Section FSAR 2.4S.3. The staff's review confirmed that the information in the application addresses the relevant information related to this subsection. The staff's technical review of this application includes the following factors:

- appropriateness of the models used in the flood safety analysis
- reasonableness of the parameters chosen in the modeling
- adequacy of the combinations of flood-causing events
- validity of the applicant's safety conclusions for potential PMF hazards at the site.

The staff determined that the models and methods used by the applicant in FSAR Section 2.4S.3 are currently used in standard engineering practices. HEC-HMS and HEC-RAS are routinely used to estimate historical and hypothetical flood hydrographs and the corresponding water surface elevations in rivers and streams. Therefore, the staff concluded that the applicant has appropriately selected numerical models to estimate the PMF and its corresponding water surface elevation in the Colorado River near the STP Units 3 and 4 site.

The staff reviewed the applicant's selection of parameters for the HEC-HMS and HEC-RAS models. The staff agreed with the applicant's determination that unit hydrograph parameters derived for a smaller rainfall event need adjustments to account for the nonlinear basin response during a PMP event. The staff determined that the applicant's approach, which follows the recommendation of USACE (1994), is acceptable. The staff also agreed with the applicant's selection of the loss-rate parameters in the HEC-HMS analysis. Setting the initial loss rate to zero will maximize the runoff generated from the PMP event and is therefore conservative. The staff determined that the continuing loss rate selected by the applicant is based on recommendations of another Federal agency (FERC, 2001) that estimates the PMF for designing and regulating critical hydroelectric dams. The staff concluded that the applicant has conservatively selected the minimum of the recommended continuing loss rates for the HEC-HMS model and thereby has maximized the produced runoff. The staff therefore found that the applicant's selection of the continuing loss-rate parameter to estimate the PMF in the Lower Colorado River Basin is acceptable. The staff also determined that the subbasin configuration and PMP storm patterns used for the HEC-HMS analysis are acceptable.

The staff reviewed the applicant's approach for specifying the HEC-RAS parameters. Because debris is expected to be carried along with a PMF, and because the PMF is expected to inundate the overbank and floodplain areas that typically have greater roughness than the main channel due to the presence of shrubs, vegetation, and other obstacles, the staff determined that the applicant's approach for increasing Manning's roughness coefficients from their baseline values in the Halff study is appropriate. The staff also determined that the adjusted Manning's roughness coefficients used in the HEC-RAS modeling of the PMF (0.042 for the main channel, 0.054 to 0.06 for the overbank, and 0.102 to 0.114 for the floodplain areas) represent a moderately rough main channel and rough floodplain areas. For example, Chow (1959) recommends that Manning's roughness coefficients should range from 0.025 to 0.060 for major streams with a regular cross section and no boulders and from 0.045 to 0.160 for a floodplain with a medium to dense brush. Therefore, the staff determined that use of these parameters would result in a conservative estimate of the elevation of the floodwater surface at the STP Units 3 and 4 site. The staff concluded, therefore, that the applicant has appropriately selected the model parameters.

The staff reviewed the applicant's use of combined events for flooding in rivers and streams as applied to the Lower Colorado River Basin. The applicant uses three combinations of PMF scenarios to determine the most critical combination of events (see Section 2.4S.3.2.4 of this report). The applicant also uses the base flow in the Colorado River near the site combined with the PMF discharge near the STP Units 3 and 4 site, as recommended in ANSI/ANS-2.8-1992 (ANS, 1992). The PMF stillwater elevation near the STP Units 3 and 4 site is significantly lower than that resulting from the upstream domino-type dam failures and the main cooling reservoir embankment breach. Therefore, the applicant does not specifically estimate wind waves for the PMF water surface elevations. The staff agreed with the applicant's statement that any wind-wave activity coincident with the PMF in the Colorado River will be smaller than that estimated for the upstream dam-failure scenario and will therefore not exceed the estimated elevation of the floodwater surface for that scenario. The staff concluded that the applicant correctly identifies the combination of events for the PMF in the Lower Colorado River Basin.

Based on the above, the staff also agrees with the applicant's analysis that the PMF in the Lower Colorado River Basin is not the controlling flooding mechanism at the STP Units 3 and 4 site. The upstream dam failure and the main cooling reservoir embankment breach scenarios resulted in higher water surface elevations. The staff describes and reviews these flood scenarios in Section 2.4S.4 of this SER. Therefore, the staff determined that the applicant's conclusions regarding the PMF in the Lower Colorado River Basin are valid.

2.4S.3.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.3.6 Conclusion

As described above, the NRC staff reviewed the FSAR to determine the adequacy of the applicant's safety conclusions regarding the regional PMF estimates at the site. The staff determined that the applicant has selected appropriate numerical models, has used data and methods commonly used in engineering practices, has conservatively selected model parameters as suggested by studies of a similar nature routinely performed by other Federal agencies, and has used combinations of events recommended in ANSI/ANS-2.8–1992 for nuclear power plant sites. Therefore, there is no outstanding information required to be addressed in this section of COL FSAR. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

As set forth above, the applicant presents and substantiates information relative to the potential for site inundation due to the PMF. The staff reviewed the available information and concluded, for the reasons given above, that the identification and consideration of the PMF in the vicinity of the site and site regions are acceptable.

The staff determined that the applicant's conclusions regarding PMF water surface elevation in the Colorado River near the STP Units 3 and 4 site are acceptable. Therefore, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL License Information Item 2.15 is adequate and acceptable.

2.4S.3.7 References

American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, Historical Technical Reference, July 1992.

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U.S. Bureau of Reclamation, “Mansfield Dam Comprehensive Facility Review, Highland Lakes Dams, Lower Colorado River Authority,” United States Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, CO, March 2003; http://www.usbr.gov/dataweb/dams/tx01087.htm; accessed by the applicant on February 20, 2007.

2.4S.4 Potential Dam Failures

2.4S.4.1 Introduction

This section of the FSAR addresses potential dam failures to ensure that any potential hazard to safety-related structures due to failure of onsite, upstream, and downstream water-control structures is considered in the plant design.

This section of the SER presents the staff's review of the estimation of the flood level caused by different dam failures. The specific areas of review are as follows: (1) dam-failure permutations, (2) unsteady flow analysis of potential dam failures, (3) water-level determination, and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

The staff reviewed two postulated dam-failure scenarios: (1) dams on the Colorado River upstream of the STP Units 3 and 4, and (2) the main cooling reservoir embankment breach. The staff identifies that the latter case is found to be the controlling scenario with water-level estimates higher than the bounding design flood level specified in the ABWR DCD, which therefore indicates the need for flood protection.

2.4S.4.2 Summary of Application

In Section 2.4S.4, the applicant addresses the site-specific information about potential dam failures. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Items 2.14 and 3.5.

COL License Information Items

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential flooding at the plant site, including flood history, flood design considerations, and effects of local intense precipitation. This information is provided below.

- COL License Information Item 3.5 Flood Elevation

COL License Information Item 3.5 requires COL applicants to ensure that the design-basis flood elevation for the ABWR standard plant structures will be 30.5 cm (12 in.) below grade. This information is provided below.

2.4S.4.3 Regulatory Basis

The relevant requirements of the Commission regulations for the potential dam failures, and the associated acceptance criteria, are in Section 2.4.4 of NUREG–0800.

The applicable regulatory requirements for identifying the effects of dam failures are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.4.4 Technical Evaluation

The staff reviewed the applicant's information in FSAR Section 2.4S.4. The staff's review confirmed that the information in the application addresses the relevant information related to the potential dam failures. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.4. This FSAR section considers the following:

- inundation due to offsite river dam failures
- inundation due to a breach of the main cooling reservoir embankment.

COL License Information Items

- COL License Information Item 2.14 Floods

- COL License Information Item 3.5 Flood Elevation

The staff's review of these COL license information items is provided below:

2.4S.4.4.1 Dam Failure Permutations

Information Submitted by Applicant

The applicant considers two permutations for upstream dam failures in the Colorado River Basin. The first permutation considers the simultaneous failure of all dams upstream of Buchanan Dam induced by a seismic event. The recommendation in ANSI/ANS-2.8–1992 (ANSI, 1992) is to use a coincidental flood, the lesser of one-half PMF and the 500-year flood, during the failure event. The recommendation is also to use a 2-year wind wave that occurs coincidentally. The applicant states that estimates of the 500-year flood discharges into the Buchanan and Mansfield dams are approximately 10,828 and 14,150 m^3/s (382,400 and 499,700 cfs), respectively. Halff Associates, Inc. (2002) estimated the standard project flood discharges for the two dams as approximately 13,728 and 20,870 m^3/s (484,800 and 737,000 cfs), respectively. The applicant has conservatively selected a coincident flood discharge of 14,158 m^3/s (500,000 cfs) for the two dams.

The second failure permutation considers a domino-type failure of upstream dams with the same coincidental wind and flood events as the first one. However, the applicant assumes the failures to occur in such a way that the combined top-of-dam storage for all dams upstream of Buchanan Dam arrives at the same time before Buchanan Dam fails. The applicant determines that the second of these two permutations would produce the larger flood because the travel and arrival times of the peak discharge are deliberately aligned to produce the largest downstream peak discharge. Therefore, the applicant only analyzes the second permutation for upstream dam breaches in the Colorado River Basin.

FSAR Subsection 2.4S.1.2.2 describes the hydrologic features in the vicinity of STP Units 3 and 4. FSAR Section 2.4S.4 describes flooding due to the postulated domino-type series of dam failures on the Colorado River. The base-case postulated floods coupled with a one-half PMF produces a peak stage of 8.7 m (28.6 ft) MSL. The site is located on the west bank of the Colorado River in Matagorda County, Texas (FSAR Section 2.4S.4). The two large main stem dams are the Buchanan and Mansfield dams, which are at river kilometers 647 and 491 (river miles 402 and 305), respectively, upstream of the site (FSAR Subsection 2.4S.4.1.1). Coupled with a one-half PMF, these dam failures produced a peak stage of 8.7 m (28.6 ft) MSL (FSAR Subsection 2.4S.4.2.1.5) in the base case. The values were lower for a sensitivity case with an increased bottom roughness.

The main cooling reservoir is a manmade reservoir enclosed by a 19.9 km- (12.4 mi)-long embankment. FSAR Subsection 2.4S.1.2.1 discusses the location and function of the main cooling reservoir. The applicant analyzes onsite floods resulting from a postulated instantaneous breach of the north segment of the main cooling reservoir embankment. The main cooling reservoir northern embankment is located about 713 m (2,340 ft) to the south of the STP Units 3 and 4 reactor buildings.

The main cooling reservoir embankment consists of rolled earth approximately 12.2 m (40 ft) high. The interior of the embankment is lined with 0.6 m (2 ft) of thick soil cement, but the outside face is only grass covered. The normal maximum operating water surface elevation in the main cooling reservoir is 14.9 m (49 ft) MSL. The applicant postulates the main cooling reservoir embankment failure mechanisms to include excessive seepage from (1) piping through the foundation of the embankment, (2) seismic activity-induced liquefaction of the foundation of the embankment, and (3) erosion of the embankment from overtopping or from wind-wave events.

The staff reviewed the applicant's response to RAI 02.04.08-01 and STPEGS UFSAR, Revision 13, Subsection 2.4.4.1.1.3. In this subsection, the applicant considers the relative likelihood of overtopping and piping failures of the main cooling reservoir embankment. In the response to RAI 02.04.08-01, dated August 27, 2008 (ML082490086), the applicant describes a freeboard analysis performed for the main cooling reservoir (for details, see Subsection 2.4.8.2.3 of the UFSAR for STP Units 1 and 2). The maximum water level in the main cooling reservoir including the setup and wave runup was reported to be 65.2 feet MSL, which was predicted to occur on the south embankment. The top of the embankment elevation at this location is 66.9 feet MSL, thus providing about 1.7 feet of freeboard above the predicted maximum water level. The applicant concludes that the overtopping for the embankment is improbable.

The northern portion of the main cooling reservoir embankment is the most critical in terms of a flood wave directed toward the STP Units 3 and 4 site. The applicant considers two breach scenarios, one to the east and the other to the west of the circulating water pipeline.

The applicant uses the HEC-RAS for the river flood routing and RMA2 (Donnell et al, 2008) for routing the flood caused by the postulated main cooling reservoir northern embankment breach. The applicant uses a revision of the Halff study HEC-RAS simulations (Halff Associates, Inc., 2002) for the river flood routing. The applicant uses a bounding calculation to estimate sediment deposition in the STP Units 3 and 4 power block area resulting from the postulated main cooling reservoir northern embankment breach.

The applicant determines the design-basis flood elevation to be 12.2 m (40 ft) MSL, which exceeds the ABWR DCD design value. Therefore, safety-related SSCs will require flood protection. FSAR Section 2.4S.10 describes the flood-protection measures. Because flood levels in the postulated breach of the main cooling reservoir were higher than the site grade, the applicant proposes a departure, STP DEP T1 5.0-1, from the certified ABWR design.

<u>*NRC Staff's Technical Evaluation*</u>

The staff reviewed the applicant's postulation of dam-failure scenarios on the Colorado River and the main cooling reservoir. The applicant uses two permutations on the Colorado River upstream and one failure scenario on the main cooling reservoir embankment. The applicant also uses the flood events to simulate the Colorado River dam-failure scenarios, as recommended by ANSI/ANS-2.8–1992. Based on the applicant's use of ANSI/ANS-2.8–1992, the staff agreed with the applicant's postulations of the dam-failure scenarios and their descriptions. The staff's review of the applicant's analysis of postulated upstream dam breaches is described in Section 2.4S.4.4.2 of this SER. The staff concluded that further analysis of the upstream dam failure was not warranted, because the main cooling reservoir was determined to yield higher peak water elevations. The staff concurred with the applicant's conclusion that the scenario with the cascading dam failure should not be considered the design-basis flood. However, the staff found that the applicant had not considered in the FSAR a main cooling reservoir breach scenario caused by the erosion of the main cooling reservoir embankment from hurricane storm surge currents. The staff's review of this combined event is described in Sections 2.4S.5 and 2.4S.10 of this SER. These sections state the staff's determination that a failure of the northern embankment of the main cooling reservoir would not be caused by surge currents from a hurricane storm.

The staff evaluated the potential for a main cooling reservoir embankment failure due to liquefaction. The staff reviewed stability assessment and soil test data that supported the STP 1 and 2 SER. In the STP 1 and 2 SER review, the staff found adequate safety factors or measures in the design and construction to control potential problem areas. The staff considered the investigations and design of the main cooling reservoir embankment, dikes, and appurtenant structures reasonable and acceptable from a geotechnical standpoint. For the STP 3 and 4 review, the staff evaluated the applicant's analysis of the main cooling reservoir liquefaction potential and the investigations of the site subsurface included in STP FSAR Section 2.5.6, which describes the geotechnical properties of the main cooling reservoir foundation soils. After considering these properties and the adequacy of the safety factors, the staff concluded that a liquefaction-induced failure is not likely. The staff also evaluated the confirmatory analysis of the liquefaction potential performed for Units 3 and 4 subsurface soils. This analysis documented that most of the subsurface soils are classified as non-liquefiable, with some limited points that can potentially liquefy. Because these points are not contiguous, the staff concluded that they do not signify an engineering stability problem. The staff found reasonable assurance that the liquefaction of main cooling reservoir foundation soils will not occur, and it is therefore unlikely that liquefaction will cause the main cooling reservoir embankment to fail.

The staff evaluated the potential for an overtopping failure mechanism of the main cooling reservoir embankment as described in SER Section 2.4S.8. The staff's evaluation did not identify any likely flooding mechanism that indicated an overtopping of the main cooling reservoir embankment. The staff concluded that there was sufficient freeboard in the main cooling reservoir embankment for the exclusion of overtopping as a plausible failure mechanism.

The staff's review determined that piping through the main cooling reservoir embankment is the most likely mechanism for failure. The rest of this SER section focuses on the staff's review of this mechanism.

2.4S.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

Information Submitted by Applicant

The applicant analyzes the upstream dam-failure scenario in the Colorado River Basin using HEC-RAS. The model configuration was based on the earlier study (Halff Associates, Inc., 2002). Several modifications to this earlier modeling effort were motivated by the need to accommodate a more severe flooding event than was previously analyzed in the Halff study (2002). Table 2.4S.4-1 summarizes the configurations of the models before and after the modification.

Table 2.4S.4-1. The Applicant's Modifications to Halff Associates HEC-RAS Model

Model Element	Halff (2002)	FSAR	Rationale
Reach length (km) / (mi)	763 / 474	666 / 414	Applicant routes only downstream of Buchanan Dam
Number of cross sections used	1048	793	Reduced reach length
Bridge crossings	Included	Not included	Assumed to have been washed away
Levees	Included	Some removed as appropriate	Represents more realistic flood propagation
Buchanan reservoir	Baseline Halff (2002)	Modified	Enlarged to accommodate aggregated initial volume of water
Flood plain geometry	Baseline Halff (2002)	Extended using USGS 30-m (98-ft) elevation data set	To allow for larger flow scenario than used in earlier study
Bottom roughness within 4 mi downstream of Buchanan and Mansfield dams[(a)]	Baseline Halff (2002)	Values increased by a factor of 2 compared to those used by Halff Associates, Inc. (2002) for Manning's n	To account for increased roughness due to dam-break debris
Bottom roughness for areas beyond 6 km (4 mi) downstream of failed dams[(a)]	Baseline Halff (2002)	Values increased by 20 percent over those used by Halff Associates, Inc. (2002) for Manning's n	To account for increased floodplain roughness due to larger extent than incorporated in earlier study (Halff Associates, Inc. 2002).
(a) The applicant refers to this scenario as the sensitivity case and compares its results with the unmodified base case.			

The applicant uses the sum of the maximum water volumes for each of the 56 impoundments upstream of Buchanan Dam as an input for the volume of the stored water in the Buchanan Reservoir to maximize the synchronized peak initial release. The applicant postulates the Mansfield Dam to fail when it overtopped by an estimated overtop depth of 0.9 m (3 ft). The applicant routes the resultant Buchanan and Mansfield dam-break flows with the addition of tributary flow and base-flood flow of 14,158 m^3/s (500,000 cfs) downstream to the river segment adjacent to the STP site.

The applicant analyzes the main cooling reservoir embankment dam failure and the resulting flood hazards using combined simulations of two models: FLDWAV (Fread and Lewis, 1998) and RMA2 (Donnell et al., 2008). The applicant simulates the outflow hydrograph from the main cooling reservoir following a postulated embankment breach using FLDWAV. The applicant then inputs the outflow hydrograph into the RMA2 model to simulate the two-dimensional flood flow outside of the main cooling reservoir embankment. The applicant then performs a bounding calculation to estimate the potential for deposition of these sediments in the STP Units 3 and 4 power block area, in order to determine the potential for an increase in the floodwater surface elevation resulting from the main cooling reservoir northern embankment breach.

The applicant assumes that large concrete structures such as STP Units 1 and 2, STP Units 3 and 4, and other tall and durable structures will remain in place during flooding following the main cooling reservoir embankment breach, while less durable structures, such as metal skin buildings and warehouses, will be mostly removed leaving only the steel framing of these structures in place. The applicant accounts for the effect of these standing structures and other debris by using a higher Manning's n value in the areas where these objects will be present. For the breach-flood modeling, the applicant assumes the bottom elevation of the main cooling reservoir to be between 4.9 and 8.5 m (16 and 28 ft) MSL, with an average bottom elevation of 6.1 m (20 ft) MSL. The applicant assumes the breach side slopes to be 1 horizontal to 1 vertical, and that the breach will expand symmetrically about the center of the breach. As an initial condition of the simulation, the applicant uses a starting main cooling reservoir water surface elevation of 15.5 m (50.9 ft) MSL, which corresponds to a conservative combined effect of a normal maximum operating main cooling reservoir water surface elevation, one-half PMP, and 2-year wind waves.

The applicant uses embankment dam breach parameters recommended for earth-filled structures by the U.S. Bureau of Reclamation (USBR) (Wahl, 1998). The applicant assumes that a service road immediately downstream of the toe of the main cooling reservoir embankment will be eroded away and the terrain further downstream of the road—at approximately 8.8 m (29 ft) MSL—will be the control for the embankment breach bottom elevation. The applicant uses empirical relationships by Wahl (1998) to estimate breach width, time to failure, and peak discharge from the breach. The applicant uses the Froehlich equation to estimate the breach width because it results in the largest estimated breach width. The breach is of a trapezoidal shape, with an average width of 127.1 m (417 ft) and a bottom width of 115.8 m (380 ft). The applicant states that Froehlich's equation results in a conservative estimate of breach width (larger than observed based on a comparison of observed and estimated breach widths of the Teton Dam) and will therefore maximize the discharge through the breach.

The applicant used the Froehlich (1995) equations to estimate the breach width and peak discharge and the MacDonald and Langridge-Monopolis (1984) approach to estimate an upper envelope of the time of failure. Empirical equations for the two methods are presented in Wahl (1998). The applicant estimates the time to failure and breach width of the main cooling reservoir embankment to be 1.7 hours and 417 ft respectively, and the peak discharge to be 1,773 m^3/s (62,600 cfs). The applicant states that the peak discharge predicted by the FLDWAV model using the breach width of 417 ft and breach time of 1.7 hours is 3,681 m^3/s (130,000 cfs) 1.7 hours after initiation of the breach, which is twice as much as that predicted by Froehlich's empirical relationships.

The applicant uses the topography of the STP site, the STP Units 3 and 4 site grading plan, and the STP Units 3 and 4 plot plan to specify the future land surface levels for the RMA2 model. The applicant sets the grade elevation at the center of the power block for STP Units 3 and 4 at 11.2 m (36.6 ft) MSL, and the elevation at the corner of the power block area at 9.8 m (32 ft) MSL. The applicant includes the reactor, turbine, control, radwaste, and service buildings and hot machine shops of all four STP units in the RMA2 model grid. The applicant also includes the UHS for STP Units 3 and 4 and the ECP of STP Units 1 and 2 in the grid. The applicant sets the southern boundary of the RMA2 grid at the northern main cooling reservoir embankment, and extends the grid to FM 521 at its northern end. The applicant selects the east and west boundaries of the RMA2 grid to be far enough away from STP Units 3 and 4 so that conditions at the model grid boundaries will have little influence on simulated variables near STP Units 3 and 4. To ensure model stability, the applicant uses an artificial sump along the eastern, northern, and western boundaries of the RMA2 model grid. The RMA2 grid extends 1,790 m (5,873 ft) in the north-to-south direction and 3,796 m (12,455 ft) in the east-to-west direction. The RMA2 grid includes 2,348 nodes and 1,088 elements varying in size from approximately 232.3 m^2 (2,500 ft^2) near the reactor buildings to approximately 13,378 m^2 (44,000 ft^2) away from STP Units 3 and 4.

The applicant specifies the Manning's n values over the RMA2 grid using published values (Arcement and Schneider, 1989; USACE, 2005). The applicant considers all buildings that are taller than 18.9 m (62 ft) MSL to remain in place during the main cooling reservoir embankment breach flood and these building will totally block the flow. The applicant assigns high roughness values to the area of the buildings that will fail due to the effects of the flood flow.

The downstream boundaries of the model grid are located sufficiently far away so that the maximum flood elevation at the STP Units 3 and 4 safety-related SSCs occurs before the flood front reaches the boundaries. The applicant uses a constant water surface elevation at the downstream boundaries.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Because the estimated design-basis flood level is higher than the site grade, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. Correspondingly, the applicant proposes flood-protection measures as described in FSAR Section 2.4S.10.

NRC Staff's Technical Evaluation

The applicant uses a combination of FLDWAV and RMA2 to simulate the flood flow following the postulated main cooling reservoir embankment failure and to estimate site characteristics related to this flood. The staff's review of the applicant's analyses considers the following factors:

- whether the applicant uses models that are appropriate for the hydrodynamic problem
- whether the applicant's parameter choices are conservative
- whether the applicant uses appropriate combinations of events
- whether the applicant correctly selects the design-basis flood and the associated site characteristics
- whether the applicant incorporates an acceptable level of conservatism to provide reasonable assurance for the protection of SSCs important to safety
- whether the applicant's results are reproducible

The staff reviewed the applicant's postulated dam failure scenarios and the applicant's use of hydrologic modeling for the Colorado River. The applicant also uses the conservative flood events to simulate the Colorado River dam failure scenarios recommended by ANSI/ANS-2.8-1992. Based on the applicant's use of ANSI/ANS-2.8–1992, the staff agreed with the applicant's postulations of the dam failure scenarios, their descriptions, and the hydrologic modeling approach. In RAI 02.04.04-14, the staff asked the applicant to describe how the breach width and time parameters were determined and to demonstrate that the most conservative plausible breach scenario is selected for the main cooling reservoir embankment. The applicant responded to the RAI 02.04.04-14 in a letter dated November 22, 2010 (ML110030201).

The staff also determined the need to update the main cooling reservoir embankment breach flood analysis to describe the sensitivity of the flood characteristics to the plausible breach widths and breach time parameters. The staff was not able to determine the characteristics of the design-basis flood at the STP Units 3 and 4 site based on the available information and therefore issued RAI 02.04.04-13. This RAI was tracked as Open Item 2.4.4-1 in the SER with open items. The applicant responded to the RAI 02.04.04-13 as RAI 02.04.04-14 in a letter dated November 22, 2010 (ML110030201). Open Item 2.4.4-1 is now closed because the staff reviewed and accepted the RAI response and the applicant's design-basis flood determination of 40-ft MSL.

The staff's review determined that the models and methods used by the applicant in FSAR Section 2.4S.4 are currently used in standard engineering practices. FLDWAV is a generalized flood routing computer program that uses an implicit finite-difference numerical solution scheme to solve the complete one-dimensional St. Venant equations of unsteady flow. FLDWAV Version 2.0.0 was released in June 2000 and has the capability to model time-dependent dam breach outflows. The staff determined, therefore, that FLDWAV is an appropriate model to use for the initial estimate of flood discharge following the main cooling reservoir embankment breach. However, because FLDWAV is a one-dimensional model, it is not appropriate for use far from the main cooling reservoir embankment, where the flow is expected to spread in two dimensions over a relatively flat terrain.

The staff determined that the applicant has appropriately selected a two-dimensional hydraulic model, RMA2. The applicant has also specified the boundary condition at the southern boundary of the RMA2 computational domain using the results from FLDWAV. RMA2 Waterways Experiment Station (WES) Version 4.5 was released in June 2008 and is a two-dimensional, depth-averaged, finite-element hydrodynamic numerical model that computes water surface elevations and horizontal flow velocities for subcritical, two-dimensional, free-surface flow fields. RMA2 solves the Reynolds form of the Navier-Stokes equations for turbulent flows and has the capability to analyze both steady and unsteady flow problems. The staff determined that RMA2 is an appropriate model to simulate the spreading flood flow following the main cooling reservoir northern embankment breach.

The staff reviewed the applicant's use of simulation models in estimating the flood following the main cooling reservoir northern embankment breach. The staff's review found that the applicant has appropriately applied the FLDWAV model to simulate the discharge hydrograph resulting from the main cooling reservoir northern embankment breach. The applicant estimates the characteristics of the main cooling reservoir northern embankment breach using a set of empirical approaches. The applicant uses conservatively selected breach characteristics predicted by the empirical approaches as input to the dynamics of breach formation in

FLDWAV. The applicant also uses the NWS BREACH model (Fread, 1991) to analyze the main cooling reservoir northern embankment breach and the resulting discharge hydrograph. The applicant uses the predictions from the NWS BREACH model as an independent check of the results from the FLDWAV simulation. The description of the staff's review and confirmatory analysis of the applicant's NWS BREACH model application appears below. After reviewing the applicant's method for specifying the bathymetry in the RMA2 model, the staff determined that the applicant has used methods and data sets that are recommended by the FLDWAV and NWS BREACH user manuals. The staff also reviewed and determined that the variably sized model grid of the RMA2 is appropriate because it uses smaller computational elements near safety-related structures, where the flow is expected to change rapidly. The staff agreed with the applicant's choices for Manning's n values because they are conservative for the expected post-construction conditions in the power block area.

The staff reviewed the combination of events used by the applicant and found that the applicant has followed the dam failure permutation that is recommendations of ANSI/ANS-2.8–1992 (ANS, 1992). Therefore, the staff agreed that the applicant's design-basis flood selection is appropriate for the STP Units 3 and 4 site. RAI 02.04.04-13 and Open Item 2.4.4-1 are therefore closed.

The staff independently ran the models (NWS BREACH and RMA2) the applicant uses in the main cooling reservoir northern embankment breach flood simulations. The staff also carried out a sensitivity analysis by varying some of the model parameters to determine whether the model results were sensitive to any parameter values. The staff's analyses confirmed that the models produced the same result that the applicant presents in the FSAR. Because the applicant selects model parameters that are recommended for use in current engineering practices, the staff concluded that the applicant's results are reproducible and therefore appropriate for the STP site. The following paragraphs provide details of the staff's independent analyses.

The staff confirmed the applicant's main cooling reservoir northern embankment breach flood discharge and its sensitivity to breach parameters using available and applicable empirical equations and the NWS BREACH model. The staff performed NWS BREACH model runs to confirm the applicant's assessment of the main cooling reservoir northern embankment failure flood at the STP Units 3 and 4 site. The staff performed sensitivity studies on the NWS BREACH model parameters.

The staff used data provided by STPNOC in the Units 3 and 4 FSAR responses to various RAIs and in technical reports prepared by STPNOC's contractors. The staff also reviewed other relevant literature (MacDonald and Landridge-Monopolis, 1984; Fread, 1991; and Froehlich, 1995). The NWS BREACH model produces estimates of the breach growth and breach outflow (hydrograph) over time that can be coupled to produce sediment flux over time. The model estimates the growth of the breach based on geometric and hydraulic properties of the embankment and geotechnical parameters of the embankment material. The staff's review determined that the FLDWAV could be used with prescribed timing parameters that specify breach growth, so that the FLDWAV-estimated discharge hydrograph and breach formation approximate those produced by the NWS BREACH model (Wahl, 2010). If the conceptual model for the subsequent flooding includes multiple or cascading breaches on a river or in a channel network, the FLDWAV would be the appropriate model for simulating the more complex flow scenario. However, in the case of the postulated breach of the main cooling reservoir northern embankment, the conceptual model consists of a single breach with no downstream channel network. Consequently, the staff determined that only the NWS BREACH model is necessary to characterize the outflow discharge hydrograph. Therefore, the staff did not use the FLDWAV to estimate the outflow hydrograph from the main cooling reservoir northern embankment breach. The staff ran the NWS BREACH model using the input file provided by STPNOC. The staff was able to reproduce the results of the NWS BREACH model reported by STPNOC in the applicant's response to RAI 02.04.04-14 (ML110030201). The technical discussion that follows provides the basis for closing RAI 02.04.04-14. The staff used several variations of the NWS BREACH model parameters to investigate the sensitivity of model predictions. Based on the NWS BREACH model sensitivity analysis, the staff selected a set of conservative parameters that is expected to result in conservative predictions of breach size and peak discharge. The staff varied the elevation at which the piping failure was initiated (Z_p), the length of the dam or main cooling reservoir northern embankment (L), the Manning's roughness parameter (n), the cohesive strength (C), the friction angle (Ø) of the embankment material, and the width of the tailwater cross section. The Manning's roughness parameter in this context refers to the characteristic of the embankment material and how that characteristic affects embankment erosion.

The staff determined that the NWS BREACH model predictions are fairly insensitive to the elevation at which the piping failure is initiated (Z_p). At a Z_p of 9.1 m (30 ft) MSL and lower, the model run did not finish because of a mathematical error that was probably a result of the Z_p being too close to the bottom of the reservoir embankment. At a Z_p of 14 m (46 ft) and higher, which is very close to the top of the initial water surface elevation in the reservoir, it appears the breach did not develop fully to erode a large portion of the embankment.

The staff determined that the NWS BREACH model predictions are very sensitive to Manning's roughness parameter n. Fread (1991) presents Strickler's equation as

$$n = 0.013\,(D_{50})^{0.67}$$

Where: D_{50} is the median grain size in millimeters (mm).

Using Fread's (1991) version of the Strickler equation gives an estimate of 0.0001 for Manning's n with a D_{50} of 0.001 mm (0.000039 in.). Strickler's equation (USGS, 2011) is presented as:

$$n = 0.015\,(D_{50})^{1/6}$$

The main difference between the Fread (1991) and USGS (2009) equations is the value of the exponent for the median grain size. The USACE (1994) also indicates that the exponent in Strickler's equation should be 1/6 or 0.167:

$$n = 0.034\,(D_{50})^{1/6}$$

The constant in the USACE (1994) equation is 0.034 for natural sediment and D_{50} is in feet.

The difference in the value of the constant could be attributed to units of measurement (USGS, 2009). Using the three variations of Strickler's equation with the median grain size of 0.001 mm (0.000039 in.) that the applicant provides, the staff obtained Manning's n values of 0.0001 (in the Fread 1991 form); 0.005 (in the USGS 2009 form); and 0.004 (in the USACE 1994 form). For comparison, the recommended lowest Manning's n values for smooth brass, Lucite, and glass channels flowing partially full are at least nearly two times greater at 0.009, 0.008, and

0.009, respectively (Chow, 1959). Therefore, the staff concluded that Strickler's equation gives unreasonably small estimates of Manning's n values for the main cooling reservoir embankment, because the embankment material is expected to form surfaces that would have a much greater roughness than that of metal or glass surfaces. The staff varied the Manning's n value from 0.001 to 0.08 to conservatively cover extreme ranges of this parameter. On the basis of NWS BREACH simulation results, the staff determined that the NWS BREACH model estimates larger peak flows for larger Manning's n values. In contrast to other uses of Manning's n where the roughness increases due to vegetation, channel meanders, and other features, this context only considered the embankment material. In the broader context, the upper bound of Manning's n may exceed 0.08. The staff's investigation determined that when only the embankment material is considered, the upper end of the range investigated by the staff is unrealistically high. The staff reviewed literature (Chow, 1959; Arcement and Schneider, 1989) to guide reasonable estimates of the base embankment material roughness (as used in the NWS BREACH model). The staff concluded that the roughness parameter should be limited to Manning's n for bare earth. For additional conservatism, this approach specifically excludes considerations such as existing vegetation, channel meanders, and existing obstructions. Arcement and Scheider (1989) suggest Manning's n values ranging from 0.012 (for flow over fine sand or concrete) to 0.07 (for flow over boulders) (see Table 1 in Arcement and Scheider [1989]). The staff used this information to select the range of Manning's n used in the staff's analysis.

The staff also determined that NWS BREACH model predictions are not sensitive to values of C ranging from 1,221 to 1,953 kg/m^2 (250 to 400 lb/ft^2). At values of C lower than 1,221 kg/m^2 (250 lb/ft^2), peak discharge and breach width increased and the time to peak decreased. However, even with a very low cohesive strength of 244 kg/m^2 (50 lb/ft^2), the embankment breach width at peak flow was approximately 156 m (512 ft).

The staff determined that the NWS BREACH model predictions are only slightly sensitive to the frictional angle. Because the east-to-west running portion of the north face of the main cooling reservoir embankment is approximately 1,311 m (4,300 ft) in length, the staff limited the dam length, L, to 1,219 m (4,000 ft). The staff's simulations showed that the NWS BREACH model predictions were not at all sensitive to L values ranging from 304.8 to 1,219 m (1,000 to 4,000 ft). The staff also evaluated the sensitivity of NWS BREACH model predictions to the length of the dam with the cohesive strength set to 488 kg/m^2 (100 lb/ft^2) and the Manning's n set to 0.08. The staff noticed that when the dam length was a limiting factor (L = 152 m [500 ft]), the NWS BREACH model predicted a washout of the entire embankment at the top, but the predicted breach did not grow wider than the length of the embankment itself. When the dam length was not a limiting factor, model predictions were not sensitive to this parameter.

The staff examined the sensitivity of the tailwater cross-sectional geometry on the NWS BREACH results. The staff used the applicant's tailwater cross-sectional geometry as the base case (bottom and top widths of 183 and 853 m [600 and 2800 ft], respectively). The staff developed six alternative cross sections that were progressively wider than the base case (with top and bottom widths for each of six alternative cross sections of (1) 305 and 853 m [1,000 and 2,800 ft]; (2) 366 and 853 m [1,200 and 2,800 ft];(3) 488 and 853 m [1,600 and 2,800 ft]; (4) 610 and 853 m [2,000 and 2,800 ft]; (5) 732 and 853 m [2,400 and 2,800 ft]; and (6) 914 and 914 m [3,000 and 3,000 ft]). The staff used a tailwater section Manning's n equal to 0.06 for all cross sections examined. For stable channels and flood plains, Arcement and Schneider (1989) suggest a Manning's n range of 0.025 to 0.032 for firm soil. Arcement and Schneider (1989) also suggest that the following additions be made to the base value of Manning's n: 0.002 to

0.010 (for vegetation), 0.006 to 0.010 (for surface irregularity), and 0.000 to 0.004 (for the flow over debris deposits). The staff used the upper value for each of these ranges to determine a Manning's n value of 0.0524. In order to account for backwater effects from the tailwater cross section, the staff determined that 0.060 was a plausible selection for the roughness of the tailwater cross section.

The staff simulated the embankment breach using NWS BREACH for each alternative case and examined the predicted breach hydrographs (peak discharge, breach width, and time to peak). The staff found that the peak discharge and breach width increased asymptotically to reach their limiting values with an increase in the width of the tailwater cross section. The staff found a limiting peak discharge of 2,915 m^3/s (102,971 cfs), a limiting breach top width of 176.8 m (580.1 ft), and a limiting breach bottom width of 151.3 m (496.3 ft). The time to peak after the other parameters reach their asymptotic values was 5.70 hrs (alternative cross-sectional cases 4, 5, and 6). The staff compared these values to the NWS BREACH case for the conservative analysis and determined that the sensitivity results of the tailwater cross section did not suggest that it was the dominant factor in the development of conservative estimates for the breach parameters. For the three cases that reached the asymptotic limit, the breach width did not attain the full width of the tailwater cross section.

Based on the sensitivity analyses described above, the staff selected a set of fewer parameters to run independent NWS BREACH simulations to conservatively estimate breach size and discharge. Because the NWS BREACH model predictions were fairly insensitive to Z_p, C, and Ø, the staff selected the values of these parameters so that they would generally be expected to result in more conservative peak discharge and time to peak parameters. The staff set the initial piping elevation at 9.8 m (32 ft) MSL, the cohesive strength at 976 kg/m^2 (200 lb/ft^2), and the friction angle at 15 degrees. The staff used Manning's n values of 0.025, 0.050, and 0.075 in the NWS BREACH simulations listed below:

- Simulation 1: n = 0.025
- Simulation 2: n = 0.050
- Simulation 3: n = 0.075

Simulation 3 yielded the largest peak flow of 3,623 m^3/s (127,929 cfs); the largest breach top and bottom widths of 175.0 and 141.3 m (574.3 and 463.6 ft), respectively; and the shortest time to peak (about 1.99 hr). The staff's use of Manning's n values as high as 0.050 was conservative, and any value exceeding 0.05 would be unrealistically high. The staff also concluded that the value could be reasonably set at an even lower value, as used in two case studies reporting the use of the NWS BREACH model (Singh, 1996). The staff concluded that the use of larger values of Manning's n for bare earth would be implausible. Therefore, the main cooling reservoir breach characteristics (peak flow of 3,623 m^3/s [127,929 cfs]; breach top and bottom widths of 175.0 and 141.3 m [574.3 and 463.6 ft], respectively; and the time to peak of 1.99 hr) predicted for Simulation 3 are conservative. The staff concluded that because none of the NWS BREACH simulations yielded an estimated breach width equal to the specified width of the tailwater cross section, the geometry of the tailwater cross section was not a limiting factor in breach growth.

The staff also compared the predictions of peak discharge from the NWS BREACH model to historical observations of dam breaches compiled by Wahl (1998). The staff's motivation for conducting a comparative analysis using historical breaches was to provide an additional confirmation that the conservative physical model simulations were realistic. The State of

Colorado recommends a similar approach to estimate dam breach parameters (State of Colorado, 2010). Using the Wahl (1998) database, the staff identified historical breaches of dams that have characteristics similar to those of the main cooling reservoir. The staff used the height of the water above the breach (h_w) and the volume stored above the breach bottom (V_w) to compare the embankments listed in Wahl (1998) with the main cooling reservoir, because these two parameters are expected to significantly affect the breach characteristics and subsequent peak discharge. The main cooling reservoir has an h_w of 6.68 m (21.9 ft) and a V_w of 1.88×10^8 m^3 (152,700 acre-ft). The staff searched the historical dam breach database to select entries with an h_w that ranged between 4 and 15 m (15 to 50 ft) and a V_w that ranged between 1.23×10^8 to 3.70×10^8 m^3 (100,000 to 300,000 acre-ft) to reasonably bound the corresponding characteristics of the main cooling reservoir. The database contains multiple entries for the same dam failure events if they were reported by several sources. The staff found 172 records that match the selected h_w range in the Wahl (1998) database. These records are associated with 59 unique failure events. Table 2.4S.4-2 shows the breach parameters listed in the database for the 172 records. The staff's review of the use of the historical database concluded that the main cooling reservoir embankment is more comparable to dams than to levees.

Table 2.4S.4-2. Parameters of Historical Dam Breaches With h_w Between 4 and 15 m (15 to 50 ft)

Parameter	Minimum	Maximum
Water height above breach bottom (h_w) (m) [ft]	4.1 [13.3]	15.2 [49.9]
Peak flow (Q_p) (m^3/s) [cfs]	29.4 [1,038]	3,115 [110,005]
Final breach top width (m) [ft]	9.2 [30.2]	153.0 [502.0]*
Final breach bottom width (m) [ft]	1.7 [5.6]	97.0 [318.2]
Average final breach width (m) [ft]	4.7 [15.4]	185.9 [609.9]*
Breach formation time (hr)	0.25	1.5
Failure time (hr)	0.5	5.0
m=meter; ft=foot; hr=hour; cfs=cubic foot per second *The maximum reported final breach top width in the database is smaller than the maximum reported average final breach width for the 172 selected records. This inconsistency exists in the database because not all breach characteristics are reported for all events.		

The database lists nine entries that include volumes above the breach bottom, V_w, in the ranges of interest. Seven of these records are associated with the Teton Dam failure, and the remaining two are associated with the Martin Cooling Pond failure. Table 2.4S.4-3 lists the parameters for the Teton Dam and Martin Cooling Pond failures.

Table 2.4S.4-3. Parameters of Historical Dam Breaches With V_w Between 1.23 x 10^8 to 3.70 x 10^8 m^3 (100,000 to 300,000 acre-ft)

Parameter	Teton Dam	Martin Cooling Pond
Water height above breach bottom (h_w) (m) [ft]	67.1–83.8 [219.9–275.0]	8.5 [28]
Volume of water above breach bottom (V_w) (m^3) [acre-ft]	3.10 x 10^8 [251,321]	1.36 x 10^8 [110,257]
Peak flow (Q_p) (m^3/s) [cfs]	65,120–65,136 [2,299,691–2,300,256]	3,115 [110,005]
Final breach top width (m) [ft]	Not available	Not available
Final breach bottom width (m) [ft]	Not available	Not available
Average final breach width (m) [ft]	151 [495]	185 [607]
Breach formation time (hr)	1.25	Not available
Failure time (hr)	4	Not available
m=meter; ft=foot; hr=hour; cfs=cubic foot per second		

The only entries in the database that meet the staff's selected range of values for water height and volume are those associated with the case of the Martin Cooling Pond embankment failure. The Teton Dam breach water height exceeds the search criteria for that parameter. Therefore, the only historical dam breach entries in the database that are similar to the postulated main cooling reservoir breach are those for the Martin Cooling Pond, which has a larger h_w of 8.5 m (28 ft) compared to the h_w value of 6.7 m (21.9 ft) of the main cooling reservoir. The Martin Cooling Pond has a smaller V_w of 1.36 x 10^8 m^3 (110,257 acre-ft) compared to the V_w value of 1.88 x 10^8 m^3 (152,700 acre-ft) of the main cooling reservoir. The final average breach width for the Martin Cooling Pond was 185 m (607 ft) compared to the NWS BREACH model-predicted main cooling reservoir average breach width of 210 m (688 ft) at the peak flow. The conservatively estimated peak flow of 3,623 m^3/s (127,929 cfs) for the main cooling reservoir exceeds the reported peak flow of 3,115 m^3/s (110,257 cfs) for the Martin Cooling Pond. On the basis of this comparison, the staff concluded that the predictions of the NWS BREACH model are reasonable and conservative for the postulated main cooling reservoir northern embankment failure. The staff found that this outcome supports the adequacy and realism of the staff's conservative use of the NWS BREACH model.

The staff also compared the NWS BREACH model results with those derived from empirical equations for the predictions of breach parameters. The staff compared the NWS BREACH model results to those obtained from the staff's use of the Froehlich (1987, 1995) and the MacDonald and Langridge-Monopolis (1984) approaches. Wahl (2004) evaluated these empirical approaches and presented prediction intervals for both empirical prediction equations. Wahl (2004) reported that the prediction interval for the average breach width and peak discharge were narrower (indicating a better fit to the data) for the Froehlich equations than those obtained using the MacDonald and Langridge-Monopolis equations. Wahl's assessment is based on a statistical analysis of the mean prediction error in breach parameter estimates (breach width, failure time, and peak discharge) for historical breaches. Wahl defined the prediction interval using log-transformed differences between the observed and the respective predicted breach parameters. To assess the goodness of fit between these methods, Wahl used minus two and plus two log-transformed standard deviations of the prediction errors. Methods with small prediction errors and associated narrower prediction intervals were assessed to have a better predictive capability. The staff reviewed the analysis conducted by Pierce et al. (2010), which concluded that the Froehlich (1995) equations were valid for conservative peak outflow predictions. Wahl (2004) concluded that the Froehlich equations had the lowest prediction error and the smallest uncertainty of all peak flow prediction techniques, including the MacDonald and Langridge-Monopolis (1984) approach.

The staff's NWS BREACH Simulation 3 results for both average width and peak discharge fall within the prediction interval of the Froelich and the MacDonald Langridge-Monopolis empirical methods. Therefore, the staff concluded that in addition to the realism support provided by the historical comparative analysis, the conservative application of the NWS BREACH model resulted in estimated breach characteristics that are supported by the empirical approaches. The staff concluded that this outcome demonstrates the adequacy of the approach that the staff used to evaluate the applicant's breach parameter estimates.

The staff determined that Simulation 3 is the most conservative of the NWS BREACH simulations. Therefore, the staff used the discharge hydrograph from this simulation as input to the RMA2 model. The staff conducted a series of RMA2 confirmatory and sensitivity analyses to evaluate the flooding at the STP Units 3 and 4 site due to a breach of the main cooling reservoir northern embankment. The postulated breach location was about 762 m (2,500 ft) away from the site. The staff's sensitivity analyses were based on the RMA2 hydrodynamic model setup provided by the applicant.

The applicant uses two postulated main cooling reservoir northern embankment breach scenarios. These two scenarios use the same breach widths (140 m [460 ft]) and peak discharge (3,653 m^3/s [129,000 cfs]), but the scenarios vary in the location of the breach on the main cooling reservoir northern embankment. They are both called east and west embankment breach scenarios.

The staff confirmed that the applicant's hydrodynamic model setup (boundary conditions) is consistent with recommendations in the literature or in the RMA2 User's Manual. The staff also determined that the applicant's values for parameters (such as Manning's roughness coefficient and turbulent exchange coefficients) for the post-construction conditions expected in the power block area are conservative and are based on values reported in the literature and in the RMA2 User's Manual.

The applicant uses an artificial sump near the open boundary of the RMA2 simulation domain to avoid model instability. The applicant states that the sensitivity analysis performed for the fixed elevation boundary condition in the artificial sump does not affect the floodwater surface elevation at the STP Units 3 and 4 site. The staff agreed that the effect of the artificial sump would not be significant, because the artificial sump is located relatively far from the area of interest where STP Units 3 and 4 safety-related SSCs would be located.

The applicant sets the open downstream boundary condition at 9.9 m (32.5 ft) MSL. The applicant also describes a sensitivity analysis in FSAR, Section 2.4S.4.2.2.4.1, which examines the effect of increasing the open downstream boundary condition to an elevation of 10.4 m

(34 ft) MSL. The applicant states that the effect on the floodwater surface elevation at the STP site because of a change in the open downstream water surface elevation is minor. The staff used the discharge hydrograph generated by the NWS BREACH model (Simulation 3 above) to specify the upstream boundary condition to the RMA2 model. The staff used two scenarios for the RMA2 simulations. The first scenario consisted of the discharge hydrograph obtained from NWS BREACH Simulation 3, with the downstream open boundary condition in the RMA2 grid set to 9.9 m (32.5 ft) MSL. In the second scenario, the staff used the same discharge hydrograph at the upstream boundary in the RMA2 grid, but changed the downstream open boundary condition to 11.0 m (36 ft) MSL to determine whether the choice of the downstream open boundary condition setting significantly affects the floodwater surface elevation at the STP Units 3 and 4 site.

The RMA2 model needs a “spin-up” before applying the breach discharge hydrograph as a boundary condition. A dynamically consistent combination of water surface elevations and flow patterns is necessary as an initial condition for RMA2; a flat water surface and no flow with water over the entire model domain is one such condition. However, before the discharge resulting from the main cooling reservoir embankment breach arrives at the upstream boundary of the RMA2 model domain, the modeled area will be dry. To reconcile the model requirement with reality, the staff initially set the water surface elevation at 20.1 m (66 ft) MSL with a small discharge. The staff then linearly decreased the water surface elevation at the downstream open boundary to an elevation equal to that used as the final open boundary condition (9.91 m [32.5 ft] MSL for the first scenario and 11.0 m [36.0 ft] MSL for the second scenario). After the “spin-up” period, the RMA2 model domain would have a small water depth with a small discharge. The staff then applied the NWS BREACH Simulation 3 hydrograph at the upstream boundary while keeping the water surface elevation constant at the downstream open boundary. The staff performed two RMA2 simulations for the east and the west breach scenarios used by the applicant.

Table 2.4S.4-4 includes the staff’s summary of the predicted water surface elevations in the RMA2 simulations at the same locations as those shown in FSAR Figure 2.4S.4-19, except for Location 7, which is near the breach and is not in the power block area. The staff noted that increasing the specified water surface elevation at the downstream open boundary results in slightly higher water surface elevations in the power block area. This increase is about 0.08 m (0.25 ft). Because the increase in water surface elevation is small and the specified elevation is conservatively chosen, the staff concluded that the effect of the chosen downstream open boundary condition is minor. The staff conducted independent RMA2 simulations and concluded that the estimated maximum water surface elevation in the power block area would be 11.9 m (39.04 ft) MSL.

Table 2.4S.4-4. East Breach Peak Flood Elevations (m [ft] MSL)

Scenarios	Locations						
	Unit 4 North	Unit 3 North	Unit 4 South	Unit 3 South	Unit 4 UHS South	Unit 3 UHS South	Between Units 3 and 4
East Breach Scenario 1 (NWS BREACH Simulation 3)	11.00 (36.09)	10.90 (35.76)	11.55 (37.90)	11.66 (38.26)	11.70 (38.39)	**11.79 (38.69)**	11.45 (37.56)
East Breach Scenario 2 (NWS BREACH Simulation 3 with open downstream boundary set to 11.0 m [36.0 ft] MSL)	11.16 (36.62)	11.13 (36.53)	11.62 (38.13)	11.74 (38.51)	11.77 (38.63)	**11.87 (38.94)**	11.50 (37.73)
West Breach Scenario 1 (NWS BREACH Simulation 3)	11.04 (36.22)	10.87 (35.66)	11.63 (38.16)	11.68 (38.31)	**11.82 (38.79)**	11.60 (38.05)	11.45 (37.55)
West Breach Scenario 2 (NWS BREACH Simulation 3 with open downstream boundary set to 11.0 m [36.0 ft] MSL)	11.18 (36.69)	11.11 (36.45)	11.70 (38.40)	11.76 (38.58)	**11.90 (39.04)**	11.70 (38.38)	11.50 (37.73)
MSL=mean sea level; m=meter; ft=foot Values in boldface indicate the maximum floodwater surface elevation for each scenario.							

Because the discharge following the postulated breach of the main cooling reservoir northern embankment is expected to carry a large amount of eroded embankment material, a significant deposition of this sediment could occur at the STP Units 3 and 4 site. The staff performed a bounding calculation to conservatively estimate a potential change in the topography of the power block area of STP Units 3 and 4 resulting from the postulated northern main cooling reservoir embankment breach. The flood would carry scoured embankment sediments and sediment from the postulated formation of a scour hole. The staff conservatively assumed that all of the combined mobilized sediment would deposit in the power block area, therefore resulting in an additive upward shift of the maximum floodwater surface elevation estimated by the RMA2 model. The staff concluded that based on geotechnical information regarding main cooling reservoir embankment foundation soils, the formation of a scour hole immediately below the main cooling reservoir embankment would be unlikely. Therefore, the staff agreed with the applicant's conclusion that the scour hole formation due to the postulated main cooling reservoir embankment breach would more likely occur downstream of the embankment, in native uncompacted soil areas, rather than in the compacted soils adjacent to the embankment.

The staff used descriptions of embankment geometry (Bechtel Energy Corporation, 1984) and the NWS BREACH model to estimate the breach width. The staff computed the volume of eroded embankment material using the NWS BREACH model-predicted final average breach width of 209.7 m (687.9 ft). The staff estimated the volume of eroded embankment sediment to be 88,103 m^3 (3,111,318 ft^3). The staff doubled the applicant's estimate of the scour hole volume of 43,693 m^3 (1,543,000 ft^3) to conservatively account for uncertainty in the dimensions of the postulated scour hole. Therefore, the staff's estimate of total volume of mobilized sediment is 175,489 m^3 (6,197,318 ft^3).

The staff postulated that in the bounding case, all of the mobilized sediment could be deposited in the power block area. The staff used FSAR Figure 2.4S.4-15 to estimate the site area where all sediment is postulated to deposit. The staff's estimated size of this area is approximately 924.8 m (3,034 ft) by 882.7 m (2,896 ft). The staff also estimated that 20 percent of this area could be covered by buildings and consequently, would not be available for deposition. Therefore, the staff estimated the area available for deposition to be 653,031 m^2 (7,029,171 ft^2). The staff also estimated that evenly distributing the total mobilized sediment volume over this area would yield a uniform thickness of 0.27 m (0.88 ft). Because the water velocity during the flood would be significant, the staff determined that a significant portion of the mobilized sediment would likely be carried beyond this area. Therefore, the assumption that all of the mobilized sediment would deposit in the selected area is conservative.

To conservatively estimate the maximum water surface elevation under the bounding sediment deposition scenario, the staff added the bounding estimate of uniform sediment deposition thickness to the maximum water surface elevation estimated in the power block area resulting from the postulated northern main cooling reservoir embankment breach. Consequently, the staff estimated the maximum floodwater surface elevation under the bounding sediment deposition scenario in the power block area to be 12.2 m (39.9 ft) MSL. Therefore, the staff determined that sediment deposition in the power block area of STP Units 3 and 4 would not result in a floodwater surface elevation that exceeds 12.2 m (39.9 ft) MSL.

2.4S.4.4.3 Water Level at the Plant Site

Information Submitted by Applicant

The highest water surface elevation during the RMA2 simulations, 11.8 m (38.8 ft) MSL, occurred at the STP Unit 4 UHS structure for the west breach scenario, approximately 1.75 hours after the breach. The peak flow velocity of approximately 1.44 m/s (4.72 fps) occurred between STP Units 3 and 4 approximately 1.75 hours after the breach. The applicant performs a sensitivity analysis by changing the downstream boundary condition from a constant elevation of 9.9 m (32.5 ft) MSL to 10.4 m (34 ft) MSL. This change does not affect the peak floodwater surface elevations at the STP Units 3 and 4 site.

The applicant selects the design-basis floodwater surface elevation of 12.2 m (40 ft) MSL at the STP Units 3 and 4 site.

Sedimentation and Erosion

The applicant estimates that the main cooling reservoir embankment will contribute approximately 48,138 m^3 (1.7 million ft^3) of clay, 2,142 m^3 (75,644 ft^3) of sand, and 3,329 m^3 (117,562 ft^3) of soil cement to the flood. The applicant also estimates that the flood following the main cooling reservoir embankment breach will produce a scour hole approximately 6.1 m (20 ft) deep, 61.9 m (203 ft) long, and 115.8 m (380 ft) wide and will therefore contribute approximately 42,475 m^3 (1.5 million ft^3) of clay to the flood flow.

The applicant estimates that the flood following the main cooling reservoir embankment breach will not cause severe erosion of concrete, asphalt, compacted gravel, and grass surfaces near the plant area of STP Units 3 and 4. Some minor erosion around the corners of buildings will be expected, but the applicant expects that the safety-related functions will not be adversely affected by this minor erosion.

In a revised response to RAI 02.04.04-15 dated November 22, 2010 (ML111150106), the applicant describes a bounding analysis of sediment accumulation in the STP Units 3 and 4 power block area resulting from the postulated main cooling reservoir northern embankment breach. The applicant uses a sediment volume estimate of 9,756 m^3 (3,433,517 ft^3) that includes contributions from the main cooling reservoir embankment and from the formation of a scour hole adjacent to the postulated breach site. The applicant doubles this sediment volume for conservatism. The applicant also assumes that the entire volume of sediment deposits evenly near the STP Units 3 and 4 plant area. The applicant identifies the dominant flow path developed within the RMA2 simulations and selects a fan-shaped area extending northward from the postulated breach location to the FM 521 Road. The applicant uses the RMA2 computation mesh covering the fan-shaped area to estimate its size and excludes areas covered by buildings. The applicant's estimates of deposition areas are 1,825,227 m^2 (19,646,580 ft^2) and 1,667,482 m^2 (17,948,623 ft^2) for the east and west breach scenarios, respectively. The applicant computes the deposition thickness for the east and west breach scenarios by dividing the sediment volume by the respective deposition areas. The applicant's estimates of sediment deposition thicknesses are 0.11 m (0.35 ft) and m 0.12 m (0.38 ft), respectively. The applicant rounds the deposition thickness upward to 0.12 m (0.40 ft). The applicant conservatively assumes that a maximum floodwater surface elevation resulting from the main cooling reservoir northern embankment breach would be raised by the deposition thickness estimate. The applicant's estimate of a maximum floodwater surface elevation resulting from the postulated breach accounting for a potential sediment deposition in the STP Units 3 and 4 power block area is 11.9 m (39.2 ft). The applicant states that this revised maximum floodwater surface elevation is below the design basin flood elevation of 12.2 m (40 ft) for STP Units 3 and 4.

<u>Hydrodynamic Forces</u>

The staff reviewed the applicant's sedimentation and erosion estimates using the SED2D model. The staff determined that the applicant did not provide sufficient information on the SED2D model. Therefore, the staff issued RAI 02.04.04-15. The applicant's response to RAI 02.04.04-15 dated March 28, 2011 (ML110890901), provides estimates of the hydrodynamic loading on plant buildings from maximum floodwater surface elevations and flow velocities. For the east and west breach scenarios, the applicant reports maximum flow velocities of 1.44 and 1.43 m/s (4.72 and 4.68 fps), respectively, at seven selected locations in the power block area (FSAR Figure 2.4S.4-19). The applicant estimates the suspended sediment concentration during the peak discharge to be 22.33 kg/m^3 (1.394 lb/ft^3). The applicant rounds the suspended sediment concentration upward to 23 kg/m^3 (1.44 lb/ft^3) and computes a sediment-laden fluid density of 1,023 kg/m^3 (63.86 lb/ft^3). The applicant uses a maximum sediment concentration of 23 kg/m^3 (1.44 lb/ft^3), along with the maximum flow velocity, to estimate the drag force on plant buildings as approximately 214.8 kg/m^2 (44 lb/ft^2).

Spatial Extent of Flooding Due to Main Cooling Reservoir Embankment Breach

Using the topographic features near the STP site, the applicant estimates that most of the floodwaters released following the main cooling reservoir embankment breach will spread out over the area bounded by FM 521. The approximate top elevation of FM 521 ranges between 8.5 and 9.1 m (28 to 30 ft) MSL. North of FM 521 and west of the main cooling reservoir, there are levees with top elevations of approximately 7.6 to 9.1 m (25 to 30 ft) MSL. The general slope near the STP site is toward the Colorado River to the east. Therefore, the applicant concludes that after the main cooling reservoir embankment breach, most of the floodwater will flow east toward the river. However, a portion of the flow will likely reach the LRS, then flow south along the west main cooling reservoir embankment, and eventually reach the Gulf Intracoastal Waterway. The applicant concludes that it is unlikely that the flood will overtop FM 521 and the levees located west of the STP site. However, if this were to happen, some flow could also reach the Tres Palacios River located west of the STP site.

Water Level at the STP Units 3 and 4 Site from Failure of Upstream Dams

Using the HEC-RAS simulation, the applicant estimates the maximum water surface elevation during the upstream dam breach as 8.7 m (28.6 ft) MSL. The applicant estimates the coincident wind waves for the upstream dam-failure scenario at the STP Units 3 and 4 site using the 2-year wind according to the methods described in the Coastal Engineering Manual (USACE, 2008). The applicant reports that an accurate estimate of fetch length for this flood scenario cannot be made, which is also documented in the STP Units 1 and 2 UFSAR. Based on topographic variations and manmade features that may limit wind effects, the applicant identifies two critical fetches: one toward the east and the other toward the northeast of the STP Units 3 and 4 site. The applicant estimates the fetch toward the east to be approximately 24.9 km (15.5 mi) long, with the maximum water depth along the fetch varying from 0.3 to 7 m (1 to 23 ft) during the peak discharge. The applicant estimates the northeast fetch to be approximately 28.3 km (17.6 mi) long, with the maximum water depth along the fetch varying from 0.3 to 2.7 m (1 to 9 ft) during the peak discharge. The applicant estimates the maximum wind setup to be approximately 1.2 m (3.9 ft). Based on the available input estimates and data, the applicant estimates the combined water surface elevation near the STP Units 3 and 4 power block area to be approximately 9.9 m (32.5 ft) MSL, with a water depth of approximately 1.4 m (4.5 ft) because the surrounding site grade around the power block and UHS is nominally 8.5 m (28 ft) MSL. The applicant concluded that because of the shallow water depth, breaking wave conditions would occur and the estimated breaking wave height would be 1.1 m (3.5 ft).

The outward slope of the power block area will be at 10 horizontal to 1 vertical. The applicant estimates the maximum wave runup to be 0.6 m (1.9 ft). Therefore, the applicant estimates that the maximum water surface elevation near the STP Units 3 and 4 power block area to be 10.5 m (34.4 ft) MSL.

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. Because flood levels from the postulated breach of the main cooling reservoir were higher than the site grade, the applicant identifies a departure, STP DEP T1 5.0-1, from the certified design. Flood protection will be needed for safety-related SSCs of STP Units 3 and 4, as described in FSAR Section 2.4S.10.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's analysis of the postulated upstream dam failures on the Colorado River. As explained below, the staff concluded that the analysis was reasonable, including the selected parameters. The applicant's conclusion, that the flood level adjacent to the site is lower than the site grade, was based on a reasonable and conservative analysis of the postulated upstream dam failures.

The applicant's FSAR did not clearly describe the spatial extent of flooding during the postulated main cooling reservoir breach. Therefore, the staff issued RAI 02.04.04-9 requesting the applicant to evaluate the spatial extent of flooding during the postulated main cooling reservoir breach and to evaluate whether the flood from the postulated main cooling reservoir breach would cause an overflow of any basin ridgelines. The applicant responded to RAI 02.04.04-9 in letters dated January 28 and February 23 of 2009 (ML090300648 and ML090710301).[1] The applicant states in the FSAR that a small portion of the flow following the postulated failure of the main cooling reservoir embankment could overflow into the Tres Palacios River, if the flood were to overtop the levees located toward the west of the STP site. The applicant states, however, that most of the flow would eventually flow to the east to the Colorado River or to the south via the LRS into the Gulf Intracoastal Waterway.

The staff reviewed the applicant's response and the main cooling reservoir embankment breach flood simulation. The staff determined that it is unlikely that a large portion of the main cooling reservoir water would cross the basin ridgelines. The depth of flow at the STP Units 3 and 4 site is approximately 1.8 m (6 ft) and becomes progressively smaller at distances farther from the main cooling reservoir embankment breach. The RMA2 simulations show that flow toward the west starts to be intercepted by the LRS and begins to turn southward. The velocity of flow in this region is less than 0.6 m/s (2 fps). Therefore, the staff concluded that it is unlikely that the flow would overtop the levees located to the west of the STP site and RAI 02.04.04-9 is closed.

The staff issued RAI 02.04.04-10 requesting the applicant (1) to discuss the composition of the flood wave (essentially a mudflow) with respect to the sediment generated from the postulated breach of the main cooling reservoir embankment and carried with the flow, including dynamic and impact forces, and to discuss the conservatism of this case compared to the case presented in the FSAR; and (2) to discuss the effects of the settlement of bank materials resulting from the postulated failure of the main cooling reservoir embankment that could result in an accumulation of a large amount of bank material at the plant site, specifically, the effects on the safety-related structures and the operation of the plant after the postulated main cooling reservoir northern embankment failure and to explain how these effects, if significant, will be addressed in Section 2.4S.14, "Technical Specifications and Emergency Operations Requirements." The applicant responded to RAI 02.04.04-10 in letters dated January 28 and February 23 of 2009 (ML090300648 and ML090710301). Subsection 2.4S.4.4.3, titled "Information Provided by the Applicant," of this SER includes a summary of the applicant's bounding calculation for sediment deposition in the STP Units 3 and 4 power block area.

To estimate sediment concentrations associated with the peak flow conditions in NWS BREACH simulations, the staff examined the change in the breach geometry and in the volume of eroded

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[1] The attachments to the letter dated February 23, 2009, which contain the applicant's RAI response, are in ADAMS Accession Numbers ML090710302 and ML090710304.

embankment material during the short period when the discharge is near its maximum. The staff converted the sediment volume to a sediment mass to estimate the sediment concentration at peak discharge. The staff estimated the sediment concentration to be 2.6 kg/m^3 (0.16 lb/ft^3), which is attributable to the contribution from the embankment but does not include contributions from the scour hole. The staff assumed that the embankment material would be dense (2,650 kg/m^3 [165 lb/ft^3]) and fully compacted (porosity = 0) to make this estimate conservative.

The staff used the applicant's estimate for the scour hole dimensions and then doubled it to account for uncertainty. The staff assumed that the scour hole was completely formed at the time of peak flow. The staff assumed a linear rise in flow to its peak value in order to compute total water volume discharged during formation of the scour hole. The staff computed the average sediment concentration during scour hole formation as the total scoured sediment volume divided by the discharged water volume. The staff's assumption of a dense scour hole material and full compaction is conservative with respect to the calculation of the scour hole contribution to the sediment concentration at peak discharge. The staff's estimate of the scour hole contribution to sediment concentration is 20.1 kg/m^3 (1.25 lb/ft^3). The staff's combined sediment concentration estimate is therefore 22.7 kg/m^3 (1.42 lb/ft^3). The staff assumed that the sediment concentration remains unchanged between the locations of the breach and power block area. The staff considered this assumption to be conservative because most of the suspended sediment would be derived from the embankment and scour hole, and because the staff doubled the applicant's estimate of the volume of sediment derived from the scour hole.

The staff conservatively estimated the density of the sediment-laden floodwater by adding the water density to the sediment concentration to obtain a combined density of 1,022.7 kg/m^3 (63.8 lb/ft^3) or an increase of 2.3 percent more than the density of water with no sediment. The staff determined that because the drag is linearly related to fluid density, it would increase 2.3 percent more than that caused by water with no sediment.

The staff examined to the RMA2 results for the main cooling reservoir west embankment breach scenario. In addition to the seven locations examined by the applicant, the staff examined maps of the velocity magnitude in the power block area and found that the maximum velocity magnitude was about 2.10 m/s (6.9 fps) when the downstream boundary condition was held at 9.9 m (32.5 ft) MSL and about 2.13 m/s (7.0 fps) when held at 11.0 m (36.0 ft) MSL. The staff found that in the RMA2 results, the velocity magnitudes were generally lower when the downstream boundary was held at a higher value. However, in some localized areas, the velocities were slightly higher.

The drag force, F, on the building wall is computed at the product of a drag coefficient, C_d, the fluid density, ρ, and the squared fluid speed, V, divided by twice the acceleration due to gravity, g:

$$F = C_d \rho V^2 / (2 g)$$

A conservative value of C_d is 2.0, freshwater has a density of 1,000 kg/m^3, and g is 9.81 m/s^2 (32.2 ft/s^2). The staff estimated that suspended sediment from the embankment breach is 2.6 kg/m^3 (0.16 lb/ft^3) and the contribution from the scour hole is 20.1 kg/m^3 (1.25 lb/ft^3). Therefore, the fluid density, ρ, is conservatively estimated as 1,022.7 kg/m^3 (63.8 lb/ft^3) or 2.3 percent larger than that of freshwater. The staff examined the RMA2 results and found that the maximum velocity in the power block area, V, in the RMA2 simulation is 2.13 m/s (7.0 ft/s). Using the above equation, a conservative value for the drag coefficient, a combined water and sediment fluid density, and a maximum water velocity, the staff computed that the maximum drag force on the power block buildings due to flooding caused by the postulated main cooling reservoir breach and subsequent flood is 485 N/m^2 (99.3 lb/ft^2).

The staff issued RAI 02.04.04-11 stating, "In response to RAI 02.04.04-9 and 02.04.04-10 (U7-C-STP-NRC-090012, February 23, 2009; Attachment 1), the applicant proposed changes to the FSAR. The proposed text for FSAR Subsection 2.4S.4.2.2.3.1 mentions that a hypothetical sump was modeled at East, West, and North boundaries. Is this configuration simply a deepening of the topography along these boundaries when the water surface elevation is held constant? How were the sumps added to the model and how were they incorporated with the specified boundary conditions? RMA2 model description suggests that these sumps were needed to improve model stability. What is the nature of the instability that is being addressed? Provide citations to publicly available references that describe this approach while using the RMA2 model."

In a letter dated August 26, 2009 (ML092430134), the applicant's response to RAI 02.04.04-11 states that a common reason for numerical instability in dynamic models is the oscillation of boundary nodes between wet and dry conditions. The applicant provides a set of references that use such an approach. The applicant also states that an artificial sump is used with the topographic elevations of nodes within the sump set to a low value so that they always remain wet. Most modeling guides recommend that the boundaries should be located far away from the region of interest, because the effects of the selected conditions at remote boundaries are less likely to affect predicted variables such as the water surface elevation in the area of interest. The applicant notes that the RMA2 model setup for the STP Units 3 and 4 site has experienced instability problems, including nonconvergence and early termination of the simulation. The applicant uses an artificial sump along the boundaries to ensure the removal of the instability. The applicant also performs a sensitivity analysis to verify that the water surface elevations set for the artificial sump will not significantly change the predicted hydraulic conditions near the STP Units 3 and 4 power block area.

The staff reviewed the applicant's response, including the references the applicant provided. On the basis of this review, the staff determined that the applicant's use of artificial sumps in RMA2 modeling is appropriate. The staff also reviewed the applicant's sensitivity analysis and agreed that the use of artificial sumps will not significantly change the flow characteristics near the STP Units 3 and 4 power block.

The staff issued RAI 02.04.04-12 stating, "In response to RAI 02.04.04-9 and 02.04.04-10 (U7-C-STP-NRC-090012, February 23, 2009; Attachment 1), the applicant proposed changes to the FSAR. The proposed text for FSAR Subsection 2.4S.4.2.2.3.2 discussed the impact of treating buildings in the main cooling reservoir breach analysis as 'hard' or 'soft.' The response states that considering the buildings as 'soft' results in a conservative estimate of flood inundation. It is not clear if this is general statement or finding from this particular model analysis. The conclusion made in the RAI response (applicant's response to RAI 02.04.04-3, in U7-C-STP-NRC-090022, Attachment 4, page 1 of 4) is not clear to the staff because removal of obstructions ('soft' buildings) may increase the cross-sectional area of the discharge even though the roughness in those areas may have been increased. Provide a discussion on why removal of 'soft' buildings would result in higher floodwater surface elevations and greater velocities."

In a letter dated August 26, 2009 (ML092430134), the applicant's response to RAI 02.04.04-12 states that the classification of buildings as "hard" or "soft" is based on an engineering judgment. The applicant also states that the removal of "soft" buildings located directly between the main cooling reservoir embankment breach and the STP Units 3 and 4 power block will also remove obstructions to flood flow and therefore, will cause a greater flood inundation. The applicant notes that the removal of "soft" buildings will make the results of the analysis more realistic.

The staff reviewed the applicant's response and determined that the removal of "soft" buildings, where the flow velocities following the main cooling reservoir embankment breach are high, would make the flooding at the STP Units 3 and 4 power block area more realistic, because some of the structures in the area were not designed to withstand the flood event caused by a postulated main cooling reservoir northern embankment breach. The staff also determined that the cross-sectional area of the removed obstructions will be relatively small compared with the cross-sectional area of the flood flow. Therefore, the staff concluded that any increase in the cross-sectional area of flow because of the removal of soft buildings would likely be minor, and the corresponding decrease in the flow velocity would also be minor. Consequently, the staff concluded that the net change in the design-basis floodwater surface elevation would also be minor.

The applicant provided a bounding calculation dated March 28, 2011 (ML110890901) to address the effects of sediment deposition at the STP Units 3 and 4 site. In addition, the staff determined that the flood induced by the postulated failure of the northern main cooling reservoir embankment has the potential to cause erosion at the site. Because the staff did not have detailed information regarding geotechnical and hydrologic properties of the post-construction top surface within and near the power block area, the staff was unable to estimate the characteristics of site-specific erosion during the flood. Therefore, the staff adopted a bounding approach and conservatively determined that the clay layer provided above the backfill material within the power block area could be eroded away. The staff postulated that infiltration of the floodwater could occur when the clay layer is eroded and the backfill material is exposed to floodwaters. Section 2.4S.12 of this SER provides an assessment of the effects of the postulated infiltration.

The staff's analysis of the postulated main cooling reservoir breach and subsequent inundation of the site involved the use of the NWS BREACH and RMA2 models. The staff's breach parameter estimates were consistent with a historical breach case and within prediction intervals of empirical approaches established in the literature. The staff determined that these outcomes establish the adequacy of its approach. The staff's independently obtained results support the applicant's conclusion for the maximum floodwater surface elevation at the STP Units 3 and 4 site, which used a different conservative approach.

As stated above in Section 2.4S.4.4.2 of this SER, the staff determined that the maximum floodwater surface elevation at the STP Units 3 and 4 site during the main cooling reservoir embankment breach event would not exceed 12.2 m (39.9 ft) MSL. Therefore, the staff determined that the applicant's design-basis flood elevation of 12.2 m (40 ft) MSL is acceptable, and RAIs 02.04.04-10, 02.04.04-11, 02.04.04-12, and 02.04.04-15 are closed. In response to RAIs 02.04.04-14 and 02.04.04-15, the applicant proposed to revise FSAR Section 02.04S.04 to clarify the process of main cooling reservoir embankment breach modeling and the effects of erosion and sedimentation on the design-basis flood level and the maximum ground water level. Revisions to the FSAR as described in the responses to RAIs 02.04.04-14 and 02.04.04-15 was tracked as Confirmatory Item 02.04.04-1. The staff reviewed FSAR Revision 6 and determined that revisions related to these issues were fully incorporated into the FSAR and therefore Confirmatory Item 02.04.04-1 is closed.

2.4S.4.5 Post-Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.4.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to estimates of the flood characteristics caused by the postulated dam break scenarios, including the main cooling reservoir embankment breach. The staff conducted independent analyses and confirmed that the applicant's design-basis floodwater surface elevation of 12.2 ft (40 ft) MSL is acceptable. The staff also reviewed the applicant's bounding calculations used to estimate the sediment deposition in the power block area as a result of the main cooling reservoir embankment breach and how this low conductivity embankment material deposition would affect floodwater surface elevations in the power block area. The staff's independent estimate of the additional increase in the floodwater surface elevation under a bounding sediment deposition scenario confirmed that the floodwater surface elevation in the power block area of STP Units 3 and 4 would not exceed 12.2 m (40 ft) MSL. The staff concluded that the surface water elevations expected during the postulated main cooling reservoir northern embankment breach event is the design-basis flood for the safety-related SSCs at the STP Units 3 and 4 site.

2.4S.4.7 References

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2.4S.5 Probable Maximum Surge and Seiche Flooding

2.4S.5.1 Introduction

This section of the FSAR addresses the probable maximum storm surge (PMSS) and seiche flooding to ensure that any potential hazard to the safety-related SSCs at the proposed site has been considered in compliance with the Commission regulations.

This SER section presents the evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: (1) probable maximum hurricane (PMH) that causes the probable maximum surge as it approaches the site along a critical path at an optimum rate of movement; (2) probable maximum wind storm (PMWS) from a hypothetical extratropical cyclone or a moving squall line that approaches the site along a critical path at an optimum rate of movement; (3) a seiche near the site and the potential for seiche wave oscillations at the natural periodicity of a water body that may affect the elevations of the flood-water surface near the site or cause a low water surface elevation affecting safety-related water supplies; (4) wind-induced wave runup under PMH or PMWS winds; (5) effects of sediment erosion and deposition during a storm surge and seiche-induced waves that may result in blockage or loss of function of SSCs important to safety; (6) the potential effects of seismic and non-seismic information on the postulated design bases and how they relate to a surge and seiche in the vicinity of the site and the site region; (7) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.5.2 Summary of Application

In Section 2.4S.5, the applicant addresses the information related to probable maximum surge and seiche flooding in terms of impacts on structures and water supply. In addition, in this section, the applicant provides supplemental information to address COL License Information Items 2.14 and 3.5 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed these issues as follows:

COL License Information Items

- COL License Information Item 2.14 Floods

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential for flooding at the plant site, including flood history, and flood design considerations.

- COL License Information Item 3.5 Flood Elevation

COL License Information Item 3.5 requires COL applicants to ensure that the design-basis flood elevation for the ABWR standard plant structures will be 30.5 cm (12 in.) below grade. This information is provided below.

2.4S.5.3 Regulatory Basis

The relevant requirements of the Commission regulations for the probable maximum surge and seiche flooding, and the associated acceptance criteria, are in Section 2.4.5 of NUREG–0800.

The applicable regulatory requirements for identifying surge and seiche hazards are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels and wave action at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

2.4S.5.4 Technical Evaluation

The staff reviewed the information in Section 2.4S5 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the probable maximum surge and seiche flooding. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented in FSAR Section 2.4S.5.

COL License Information Items

COL License Information Item 2.14 Floods

COL License Information Item 3.5 Flood Elevation

The staff reviewed the applicant's information in FSAR Section 2.4S.5. The staff found the methods and tools used in conjunction with or developed using this information to be reasonable. This section considers the following:

- inundation of the STP Units 3 and 4 site from a probable maximum storm surge (PMSS)
- effects of PMSS inundation on the main cooling reservoir embankment

2.4S.5.4.1 Probable Maximum Winds and Associated Meteorological Parameters

Information Submitted by Applicant

The applicant establishes the probable maximum meteorological winds (PMMWs) using guidance found in NOAA NWS Report 23 (NOAA, 1979). A summary of the applicant's PMMW parameters is provided in Table 2.4S.5-1 below. These values are reported in FSAR Section 2.4S.5.1 and Table 2.4S.5-2 (STPNOC, 2007).

Table 2.4S.5-1. Parameters of Probable Maximum Meteorological Winds

Parameter (units)	Symbol	Range of Values
Peripheral pressure (cm/in. of Hg)	P_w	76.50 / 30.12
Central pressure (cm/in. of Hg)	P_o	66.52 / 26.19
Pressure differential (cm/in. of Hg)	$P = P_w - P_o$	9.98 / 3.93
Radius of maximum winds (nautical miles)	R	5 to 21
Forward speed (knots)	T	6 to 20
Hg = mercury; in. of Hg = one-thirtieth of atmospheric pressure (e.g., 0.49 psia).		

Using the above characterization of the PMH, and following the guidance of NWS Report 23 (NOAA, 1979), the applicant estimates that the PMMW speed range for a stationary hurricane is 68.0 to 71.5 m/s (152 to 160 mph).

The staff issued RAI 02.04.05-6 requesting the applicant to indicate whether any effort was made to adjust the estimated PMH parameters, because more recent hurricanes have occurred since the publication of the NOAA NWS 23 report. The applicant's response to RAI 02.04.05-7, dated August 12, 2008 (ML082270381), and response to RAI 02.04.05-6, dated September 4, 2008 (ML082530449), refers to a recent NOAA analysis indicating that the period between 1945 and 1970 is considered to be a hurricane period as active as hurricane periods in the most recent decades. The applicant concludes that because the 1945 through 1970 period is covered by the analysis in the NOAA NWS 23 report, the estimated PMH in NWS 23 will provide a conservative assessment and will account for any increase in hurricane strength due to future climate variability.

NRC Staff's Technical Evaluation

The staff used NWS 23 (NOAA, 1979) to independently estimate the PMMW for the STP site. The staff's estimates of the PMH parameters using NWS guidance (Jelesnianski et al., 1992) are given in Table 2.4S.5-2 below. The staff also used the NOAA hurricane database and other currently available information to assess the relative severity of the NWS 23 PMH. NWS 23 covers 1871 to 1978, and the staff determined that fifty-four hurricanes have impacted Texas between 1851 and 2008 with 18.5 percent occurring outside the NWS 23 reporting period. No hurricane greater than a Category 4 has ever made landfall in Texas, and all Category 4 hurricanes impacting Texas occurred within the NWS 23 reporting period. Only 17 percent of all hurricanes in the United States occurred after the NWS 23 reporting period. Looking at the twelve most intense hurricanes to hit the United States, only three occurred outside of the NWS 23 reporting period. Therefore, the staff determined that the applicant's use of NOAA NWS Report 23 (NOAA, 1979) to derive the PMMWs is reasonable and conservative.

In regard to climate change, studies of tropical cyclone variability in the North Atlantic region reveal large interannual and interdecadal swings in storm frequency, which are linked to regional climate phenomena such as the El Niño/Southern Oscillation; the stratospheric quasi-biennial oscillation; and multi-decadal oscillations in the North Atlantic region. Recent research examining Atlantic hurricanes and climate change has focused on whether the increase in hurricane activity in the basin since the 1970s portends future large increases in a warming climate. One analysis of projected climate changes over the tropical Atlantic region during the 21st century is premised on 18 different climate models developed for the IPCC Fourth Assessment Report. A notable finding is the vertical wind shear (the difference in wind direction and speed between the lower and upper atmosphere), which is projected to increase across much of the Caribbean in the warmer climate. This factor tends to suppress the development and intensity of tropical storms and hurricanes.

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 70.5 and 66.9 m/s (157.6 and 149.7 mph) (Category 5 and Category 4), respectively. The estimated stationary hurricane wind speed of 66.9 m/s (149.7 mph) is consistent with but slightly lower than the applicant's estimated range of 68.0 to 71.5 m/s (152 to 160 mph) (Category 5) in FSAR Table 2.4S.5-3 (STPNOC, 2007).

The applicant initially used the SURGE and the NOAA Sea, Lake, and Overland Surges from Hurricane (SLOSH) models to analyze storm surges. The staff issued RAI 02.04.05-1 requesting the applicant to provide the SURGE model code and input and output files used to estimate the PMSS at the coast near Matagorda, Texas. The applicant responded to the RAI in a letter dated June 26, 2008 (ML081970231). The staff then performed an independent analysis using the applicant's implementation of the SURGE model and confirmed the applicant's analysis. Therefore, RAI 02.04.05-1 is closed.

Table 2.4S.5-2. The Staff's Estimates of PMH Parameters

Parameter (units)	Value	Source in NOAA (1979)
Latitude (degrees North)	28.6	
Coriolis parameter f (1/s)	7.1×10^{-5}	
Coastal distance (km / nautical mile)	601.9 / 325	Figures 1.1 and 1.2
Central pressure P_o (cm / in. Hg)	66.52 / 26.19	$P_w - \Delta P$
ΔP (cm / in. Hg)	9.98 / 3.93	Figure 2.3

Peripheral pressure P_w (cm / 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 / knot)	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 (m/s / mph)	70.5 / 157.6	Equation 2.2
Stationary hurricane gradient velocity (m/s / mph)	66.92 / 149.7	Equation 2.4
m=meter; ft=foot; s=second; km=kilometer; mph=mile per hour; mi=mile; cm=centimeter; in.=inch; Hg=mercury		

The staff issued RAI 02.04.05-4 requesting the applicant to explain (1) how NOAA's SLOSH Maximum of Maximum (MOM) water-level predictions were extrapolated to account for the PMH conditions; (2) whether the PMH used in this extrapolation was the same as the PMH used in the SURGE analysis to estimate the PMSS at the coast near Matagorda, Texas; and (3) how the applicant verified that the extrapolation is valid and conservative. The applicant responded to RAI 02.04.05-4 in a letter dated September 10, 2008 (ML082560248).

For item 1, the applicant describes how it uses the SLOSH MOM water levels to extrapolate to the PMH condition using a third-order polynomial curve fit. The applicant uses the NOAA pre-computed Categories 1 through 5 SLOSH MOM values with the corresponding pressure differentials in the curve fit. The applicant estimates the surge from the PMH using the difference between the peripheral and the central pressures as the predictor variable in the polynomial equation. The applicant's response provides the curve-fit procedure and describes its use. The staff verified the applicant's results using the PMH pressure differential.

For item 2, the applicant verifies that the conditions used for the SURGE application are consistent with those described in FSAR Subsection 2.4S.5.1. However, the applicant differentiates its application of SURGE with the use of the extrapolation based on the SLOSH MOM water levels, which differ in terms of the hurricane forward speed and the radius to the maximum winds. The applicant's assessment maintains that these differences are not important.

For item 3, the applicant's assessment of the conservatism of the SLOSH extrapolation is based on the fact that the extrapolated value is larger than a similar assessment made using the SURGE model. Also, the applicant states that NUREG–0933 refers to the SURGE as a conservative model.

The staff reviewed the applicant's response and determined that the applicant's extrapolation based on the NOAA pre-computed the SLOSH Categories 1 through 5 MOM may not yield conservative estimates of peak water levels at the site, because there is no physical basis for choosing the extrapolation equation that the applicant uses. Therefore, the staff independently estimated the PMH water surface elevations at the STP site using the SLOSH model and found that the surge level simulated by the SLOSH model is higher than the applicant's initial SURGE model estimate. Therefore, the staff issued RAIs 02.04.05-10 and 02.04.05-11. In response to RAIs 02.04.05-10 and 02.04.05-11, dated July 27, 2010 (ML102100047), the applicant performed storm surge simulations using the SLOSH and the USACE Advanced Circulation (ADCIRC) models. A summary of the applicant's analyses and the staff's subsequent review is described in Section 2.4S.5.4.2 below. Therefore, RAI 02.04.05-4 is closed.

2.4S.5.4.2 Surge and Seiche Water Levels

Information Submitted by Applicant

The ABWR DCD Section 2.1 requires that the design-basis flood elevation shall be no higher than 0.3 m (1 ft) below site grade; the site grade is 10.4 m (34 ft) MSL. The applicant's estimate of the storm surge water surface elevation resulting from a PMH is 9.5 m (31.1 ft) MSL, which is lower than site grade of 10.4 m (34 ft) MSL and the design-basis flood level of 12.2 m (40 ft) MSL.

The applicant estimates the PMH using NWS Report 23 (NOAA, 1979), as described in Subsection 2.4S.5.4.1 above. The applicant's procedure accounts for several factors that control the PMSS water surface elevation, but it does not include an initial sea level rise and the astronomical tide levels associated with the PMH. The applicant added these initial sea levels separately to the estimated storm surge water levels. The applicant uses an initial sea level rise of 0.73 m (2.4 ft) and a 10 percent exceedance astronomical high tide of 0.67 m (2.2 ft).

The applicant describes historical hurricane surge elevations along the Texas coastline. The applicant states that the peak storm surge elevation for a site close to STP Units 3 and 4 is approximately 4.9 m (16 ft) MSL.

The applicant initially uses two approaches to estimate the storm surge flooding elevations near the STP site. The first approach uses the SURGE model (Bodine, 1971) to estimate the storm surge at the Gulf coast near the STP site. The applicant's analysis examines a range of values for wind and bottom frictions, PMH geometries, and track speeds. The applicant increases the maximum SURGE estimates at the coast to 6.1 m (20.04 ft) MSL to account for the sea-level rise of 0.59 m (1.93 ft) due to global climate change. To estimate the storm surge level near the STP site, the applicant uses both the HEC-RAS model and the SURGE result to specify boundary conditions for a Colorado River backwater calculation. The HEC-RAS model simulates the combined effect of a 100-year river flood event combined with the SURGE results.

In the second approach, the applicant extrapolates archived results from NOAA's SLOSH model (Jelesnianski et al., 1992) runs that use several hurricane scenarios involving Category 1 through Category 5 hurricanes to account for PMH conditions near the STP site. NOAA reported the maximum water surface elevations from the suite of SLOSH runs in this archive. The archived SLOSH results included a 0.6-m (2.0-ft) sea level rise in the simulations. Although the archived SLOSH results cover a range of hurricanes, the most extreme of these is weaker than the PMH. None of the archived SLOSH results indicates the inundation of the STP site. Therefore, the applicant extrapolates these SLOSH results to estimate the PMH water surface elevations, which includes the aforementioned 0.6-m (2.0-ft) sea level rise offset. The applicant makes adjustments to this surge elevation to account for a long-term sea-level rise (0.59 m [1.93 ft]), an initial sea-level rise (0.73 m [2.4 ft]), and astronomical tides (0.67 m [2.2 ft]).

The resulting water surface elevations, the site grade elevation, and the ABWR DCD site parameter are as follows:

- HEC-RAS backwater using the SURGE water level: 7.4 m (24.29 ft) MSL.
- SLOSH extrapolation using Categories 1 through 5 estimates yields a surge water level of 8.3 m (27.2 ft) MSL.
- The consideration of a large value (10 percent exceedance astronomical tide), sea-level rise, and atmospheric pressure correction adds 1.4 m (4.53 ft) and yields a peak surge estimate of 9.7 m (31.7 ft) MSL.
- The site grade elevation is 10.4 m (34.0 ft) MSL.
- The ABWR DCD compliance elevation is 10.1 m (33.0 ft) MSL.

During the site audit conducted on August 31 and September 1, 2010, the applicant presented a summary of the SLOSH and ADCIRC analyses. On the basis of the applicant's presentation at the site audit, the staff determined that the applicant had not shown that the ADCIRC model results account for the most conservative and plausible PMH scenario because, at that time, the applicant had only simulated one PMH scenario using the ADCIRC model. Furthermore, the descriptions and results of these model applications were not in the FSAR updates.

After the site audit, the staff issued Supplemental RAI 02.04.05-11 requesting the applicant to provide additional information regarding (1) a detailed description of the ADCIRC model, including the wind-wave submodel; (2) a detailed description of supporting data sets, including the topographic and bathymetric grids; (3) a list of conservatively selected plausible PMH scenarios consistent with the NWS 23 ranges of the PMH parameters used as inputs to the ADCIRC; (4) a description and justification of why other plausible PMH scenarios were not selected as conservative; (5) a description of the sensitivity of the ADCIRC-simulated PMSS to the PMH parameters including the radius to maximum winds, forward speed, track direction, and the landfall location; (6) a description of nonlinearity in the estimated PMSS corresponding to various combinations of PMH parameters; (7) the selected PMSS near the STP site, including the wind-wave runup; (8) a detailed description of various methods used to estimate current velocities during a PMSS event; (9) a detailed description and justification of the simplifying assumptions; (10) conservatively selected current velocities and the durations that these currents will affect the main cooling reservoir embankment; and (11) relevant citations to support a justification for the ability of the grass-lined outer face of the northern main cooling reservoir embankment to withstand the current velocities without erosion severe enough to cause an embankment breach. The RAI response dated November 22, 2010 (ML103330369), states that the applicant had performed ADCIRC simulations in addition to the scenario presented to the staff during the site audit on August 31 and September 1, 2010.

The applicant's response to RAI 02.04.05-11 part (1) describes the ADCIRC model. The applicant states that the ADCIRC is a hydrodynamic circulation model that simulates water levels and current over an unstructured domain. The model is capable of a two- or three-dimensional representation of hydrodynamics using equations of motion for a moving fluid over the surface of the rotating earth. The model uses finite element and finite difference formulations for discretizations in space and time, respectively. The applicant states that the ADCIRC can handle a variety of boundary conditions, including external and internal barrier overflow and the outward radiation of waves. The unstructured computational grid allows for smaller grid elements in areas where greater spatial resolution is necessary to capture topographical variations or to accurately capture rapid changes in hydrodynamics. The model also allows for a variation in friction with the depth of flow. The applicant states that the spatially varying friction was used for low-velocity deeper offshore waters, shallow near-shore waters, rivers and inlets where velocities are expected to be higher, and in the remaining areas of the domain. The model also represents the wetting and drying of the grid elements based on computed depths at all nodes of a grid element. The model includes only wet elements, with all nodes simulated to have a positive water depth in the solution. The model uses a minimum water velocity as a criterion for determining whether water can flow from an adjacent wet element to a dry one.

The applicant states that the ADCIRC model uses the asymmetric Holland wind model (Holland, 1980). The applicant uses the USGS National Land Cover Data Classification map and land roughness lengths derived from the Federal Emergency Management Agency (FEMA) HAZUS software program, which is used to assess hazard losses—including those from hurricanes. The roughness of an inland grid element changes as the element becomes inundated during the hurricane event. The applicant carried out an extensive validation of the ADCIRC predictions on the Texas coastline for historical hurricanes. The applicant notes that these validation studies included Hurricanes Rita and Ike, which produced large storm surges and for which accurate measurements of hurricane properties and surge were available.

The applicant states that the ADCIRC uses the computer program SWAN to estimate the wind setup. The Delft University of Technology developed the SWAN program, which computes random, short-crested, wind-generated waves in near-shore and inland waters. The SWAN model accounts for wave propagation, shoaling, reflection, refraction, frequency shifting, wave interactions, white capping and breaking, and dissipation. The applicant states that water levels and currents are computed by the ADCIRC with input into SWAN, which recalculates the water depth to account for the wave processes. The ADCIRC model further uses the modified hydraulic properties computed by SWAN.

The applicant also states that along the coastal areas of the United States, FEMA has certified the ADCIRC for use in the development of Flood Insurance Rate Maps that need to account for flooding from storm surges. The applicant also notes that the ADCIRC is the standard coastal model used by the USACE.

In the response to RAI 02.04.05-11 part (2), the applicant provides a detailed description of data sets used with the ADCIRC. The applicant states that topographic data used in the ADCIRC are the most accurate and current. The applicant also states that the most accurate topographic data are derived from the Light Detection and Ranging (LIDAR) data sets from the Texas Water Development Board (TWDB) (Harris County Flood Control District, FEMA, Louisiana State University) and the Louisiana Oil Spill Contingency Office Atlas. The applicant states that the LIDAR data were initially available at a 10-m resolution and later at a 1-m resolution; the data include small-scale features such as levees, riverbanks, and roads. The ADCIRC computational grid was initially built using the 10-m (33-ft) LIDAR data and was later refined using the 1-m (3-ft) LIDAR data to include hydraulically relevant features. The alignment of major topographic features including roads, shorelines, and rivers was checked against aerial photographs and satellite images.

The applicant states that Texas topographic grid Version 13 (or the TX2008 model) incorporates the western North Atlantic Ocean, the Gulf of Mexico, the Caribbean Sea, and the Texas coastal floodplains to allow full dynamic coupling between oceans, continental shelves, and coastal floodplains. The applicant states that the TX2008 model domain's eastern boundary is the open ocean that lies along the 60°W meridian. The open ocean boundary (1) is located in deep ocean, (2) lies outside of any resonant basins, (3) is geometrically simple, (4) has limited nonlinear energy because of the depth, and (5) its tidal response is mainly determined by astronomical variations. The applicant states that the specification of a boundary condition along this open ocean boundary is simple because the hurricane storm surge response along it is mainly an inverted barometric pressure effect directly correlated to the hurricane pressure field.

The applicant also states that the TX2008 model domain is bounded by the land boundary of the eastern coastlines of North, Central, and South America. The highly detailed region represented in the TX2008 model extends from Brownsville to Port Arthur, Texas; the TX2008 model extends inland and runs along the 9- to 23-m (30- to 75-ft) elevation contour. The model incorporates the Brazos, Nueces, and Rio Grande rivers and major dredged navigation canals such as the Gulf Intracoastal Waterway; all significant levee systems, elevated roads, and railroads are barrier boundaries. The applicant notes that the grid resolution in the TX2008 model varies from 19 to 24 km (12 to 15 mi) in deep ocean and about 30 m (100 ft) in near-shore areas of Texas.

The applicant also states that the bathymetric data for the western North Atlantic, Gulf of Mexico, and Caribbean Sea were derived from the raw bathymetric sounding database from the NOAA National Ocean Service Digital Nautical Charts database and NOAA ETOPO5 data. The bathymetry for inland waterways in coastal regions of Texas was derived from regional bathymetric and dredging surveys from the USACE, NOAA, TWDB, or nautical charts. The geometry, bathymetry, and topography in the TX2008 model represent post-Hurricane Ike conditions.

The applicant states that the ADCIRC computational grid should account for pronounced vertical features that are small in the horizontal scale compared to the grid spacing. Some of these features can be a significant obstruction to the flow. Therefore, features higher than 3 m (10 ft) from the surrounding area were carefully incorporated into the model as subgrid scale weirs or lines of nodes with crown elevations.

The response to RAI 02.04.05-11 part (3) states that the applicant used combinations of three landfall points and NWS 23 PMH parameters—radius to maximum winds, approach direction, and forward speed—to specify 81 PMH scenarios that may occur at the STP site. The applicant states that NWS 23 ranges of PMH parameters near the STP site include a radius to maximum winds of 9.7 to 33.5 km (6.0 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of 11.1 to 35.1 km/hr (6.9 to 21.8 mph). The applicant notes that storm surge simulations using the SLOSH PMH extrapolation indicate that the maximum water surface elevation near STP Units 3 and 4 would be produced by a PMH scenario with a large radius to maximum winds, fast forward speed, and prevailing winds blowing from the east toward the site. The applicant concludes that the PMH would result from a storm with a radius to maximum winds of 33.5 km (20.8 mi), an approach angle of 143.8 degrees clockwise from the north, and a forward speed of 35.1 km/hr (21.8 mph, 18.9 kt).

The applicant postulated a series of hurricane scenarios using the ADCIRC to determine the maximum water surface elevation at the STP Units 3 and 4 site. The applicant used a radius to maximum winds of 38.6 km (24 mi, 21 nautical miles [nmi]); an approach direction of 135 degrees clockwise from the north; a forward speed of 37 km/hr (23 mph, 20 kt); a central pressure of 887 milibars (mb) (26.19 inches of mercury [in. Hg]); and a peripheral pressure of 1,020 mb (30.12 in. Hg). The only variables were the distance of the storm track from the site and the track's direction. The applicant used seven ADCIRC scenarios (summarized in Table 2.4S.5-3 below). In the response to RAI 02.04.05-10 (ML102100047), the applicant states that the initial conditions for the ADCIRC runs consisted of a water surface elevation that accounted for a 10 percent exceedance high tide, initial rise, and long-term sea-level rise estimated by NOAA.

Table 2.4S.5-3. Applicant's PMH Scenarios for ADCIRC Simulations

Scenario	Distance from Site	Track Direction	Maximum PMSS Water Surface Elevation
1	19.3 km (12 mi, 10.4 nmi)	NW	8.1 m (26.5 ft) MSL
2	38.6 km (24 mi, 20.9 nmi)	NW	8.9 m (29.3 ft) MSL
3	57.9 km (36 mi, 31.3 nmi)	NW	8.7 m (28.5 ft) MSL
4	38.6 km (24 mi, 20.9 nmi)	N	7.6 m (25 ft) MSL
5	38.6 km (24 mi, 20.9 nmi)	N-NW	8.8 m (29 ft) MSL
6	38.6 km (24 mi, 20.9 nmi)	W-NW	7.9 m (26 ft) MSL
7	38.6 km (24 mi, 20.9 nmi)	W	6.1 m (20 ft) MSL
N=north; NW=northwest; NNW=north-northwest; W=west; WNW=west-northwest; m=meter; ft=foot; MSL=mean sea level; nmi=nautical mile			

The applicant also states that the ADCIRC simulations use the same wind profile that the SLOSH uses because the SLOSH wind profile results in greater wind speeds than in the Holland profile for the same gradient wind speed and distance from the storm's center.

The response to RAI 02.04.05-11 part (4) states that the applicant selected PMH scenarios that represent the most conservative combination of storm scenarios, because the selected storm scenarios use the greatest ∆P that results in a stronger storm, the greatest radius to maximum winds (Scenario 2) that results in a larger storm, the greatest forward speed that increases surge heights, maximum sustained wind speed that remains constant until landfall, tracks that are least resistant to wave build-up, and a conservative wind profile.

In the response to RAI 02.04.05-11 part (5), the applicant reports the maximum surge heights predicted by the ADCIRC for the seven PMH scenarios. These maximum surge heights are listed above in Table 2.4S.5-3. The applicant notes that the ADCIRC did not successfully simulate scenario 7. The applicant estimated the surge water surface elevation at the site for scenario 7 based on a completed ADCIRC simulation that used a lower wind speed and an estimate of the incremental surge expected for the difference in wind speed. Based on the ADCIRC-simulated maximum water surface elevation at the site, the applicant concludes that the greatest storm surge occurs when the storm passes the site at a distance equal to the radius of maximum winds and the storm track direction is generally to the northwest. In a comparison of topographical data used in the SLOSH and ADCIRC, the applicant notes that the TX2008 model accounts for pronounced vertical features with a small horizontal extent like the levee surrounding the City of Matagorda and the Gulf Intracoastal Waterway.

In the response to RAI 02.04.05-11 part (6), the applicant states that to a limited degree, surge elevations do not vary linearly with track direction or distance from the site. The applicant also states that it was difficult to describe the nature of the nonlinearity, although the outcomes were consistent with the behavior of hurricane storm surges in the western Gulf of Mexico.

The applicant's response to RAI 02.04.05-11 part (7) states that based on ADCIRC simulations using the SLOSH wind profile, the estimated PMSS at the STP Units 3 and 4 site is 8.9 m (29.3 ft) MSL. The applicant also states that this PMSS would occur as a result of a hurricane traveling in a northwestern direction and passing within 38.6 km (24 mi) of the site. Until landfall, the hurricane would have a constant speed of 37 km/hr (23 mph), a central pressure of 887 mb, and a maximum sustained wind speed of 296 km/hr (184 mph, 160 kt). The hurricane's strength would gradually decay after landfall.

The applicant's responses to RAI 02.04.05-11 parts (8) through (11) are relevant to the staff's review in Section 2.4S.10 of this SER, which is where the applicant's responses and the staff's review are summarized.

NRC Staff's Technical Evaluation

Applicant HEC-RAS and SURGE Analysis

The staff issued RAI 02.04.05-1 requesting the applicant to provide the input and output files of the HEC-RAS analysis to estimate backwater effects corresponding to the PMSS estimates using the SURGE model. The applicant responded in a letter dated June 26, 2008 (ML081970231), and provided the input and output files. The staff did not use these files because the staff's independent analysis of the PMH storm surge estimate using the SLOSH model is more conservative. Therefore, RAI 02.04.05-1 is closed.

The staff issued RAI 02.04.05-2 requesting the applicant to explain why a wind-stress correction factor of 1.1 was used when, as stated in FSAR Subsection 2.4S.5.2.3.1, page 2.4S.5-4, "the stresses introduced into the air by the drops can be 10-20% of the wind stress." The applicant responded in a letter dated August 27, 2008 (ML082490086), and states that the wind-stress factor is consistent with RG 1.59. The staff determined that the applicant's response is satisfactory. Therefore, RAI 02.04.05-2 is closed.

The staff issued RAI 02.04.05-3 requesting the applicant to explain why the HEC-RAS backwater analysis was not carried out for the LRS through the Palacios Bay. The applicant responded in a letter dated August 27, 2008. This response states that because the LRS is tidal—with no upstream inflow—and is assumed to be inundated by a PMH surge, no backwater calculations were warranted. The staff agreed with the applicant's assessment. The staff's independent PMH storm surge estimate using the SLOSH model showed that the Palacios Bay would be completely inundated during the PMH event. RAI 02.04.05-3 is therefore closed.

The staff issued RAI 02.04.05-5 requesting the applicant to explain why the PMH determined from the NOAA NWS 23 report was not used as input to run the SLOSH model to estimate water surface elevations for the PMSS. The applicant responded in a letter dated September 4, 2008 (ML082530449). This response states that the SLOSH model was not publicly or commercially available for conducting an analysis specific to the PMH. The applicant's PMSS assessment is based on the NOAA precomputed SLOSH simulations for Categories 1 through 5 hurricanes. The applicant uses a third-order polynomial equation to estimate a relationship between the storm surge water surface elevation and the hurricane central pressure difference, as described above. Therefore, RAI 02.04.05-5 is closed.

As mentioned in Section 2.4S.5.1, the staff issued RAI 02.04.05-6 requesting the applicant to indicate whether any effort was made to adjust the estimated PMH parameters, because more recent hurricanes have occurred since the publication of the NOAA NWS 23 report. The applicant responded in a letter dated September 4, 2008, and refers to a recent analysis by NOAA indicating that the period between 1945 and 1970 is considered a hurricane period that was as active as hurricane periods in the most recent decades. The applicant concludes that because the 1945 through 1970 period is covered by the analysis in the NWS 23 report, a PMH estimated by the NWS 23 will provide a conservative assessment and will account for any increase in hurricane strength due to future climate variability. The staff used other currently available information to assess the relative severity of the NWS 23 PMH, as described below. Therefore, RAI 02.04.05-6 is closed.

The staff issued RAI 02.04.05-9 requesting the applicant to provide a physical basis to justify why the maximum of the maximum envelope of water surface elevation–ΔP relationship used by the applicant is valid, with a citation to an accepted and validated method that uses such a relationship, or provide a justification with a citation indicating why estimating parameters of a third-degree polynomial relationship from five data points would result in an accurate estimation of the model parameter values. The applicant responded in a letter dated September 16, 2009 (ML092610376). The applicant's response cited NUREG–0933, "A Prioritization of Generic Safety Issues - Item C-14: Storm Surge Model for Coastal Sites (Rev. 1)," dated 2007 and argued that a bathystrophic model, SURGE, is adequate for calculating design-basis water levels. The applicant repeated its response to RAI 02.04.05-4 regarding justification of the extrapolation equation. The staff reviewed the applicant's response and determined that the extrapolation procedure may not be conservative. Therefore, the staff performed an independent assessment using the SLOSH model to estimate the PMH inundation level at the site. The staff's approach is described below for the closure of RAI 02.04.05-9. In addition, the complex coastline, built-up areas, interacting streams, channels and canals, bathymetry, and topography near the STP site require that a more advanced method be used to accurately estimate the storm surge from severe hurricanes.

NRC Staff SLOSH Analysis

Because the staff determined that the applicant's PMSS estimates using the SURGE model and the extrapolation approach may not be conservative, the staff carried out independent SLOSH simulations using a range of input values that represent the variability of PMH conditions at the STP site. The NWS 23 specifies ranges of PMH parameters, radius to maximum winds (9.7 to 33.5 km [6 to 20.8 mi]), the approach direction (97.5 to 190 degrees clockwise from the north), and the forward speed of the storm (3.1 to 9.8 m/s [6 to 19 knots]). The staff used combinations of these three parameters in addition to three different landfall points to specify several PMH scenarios that may occur at the STP site. Three individual values were selected for each of these scenarios and, therefore, the staff's analysis resulted in the SLOSH simulation of a total of 81 PMH storm tracks.

The staff set the radius to maximum winds to 9.7, 20.8, and 33.5 km (6, 12.9, and 20.8 mi); the approach angle to 97.5, 143.8, and 190 degrees clockwise from the north; and the forward speeds to 3.1, 6.4, and 9.8 m/s (6, 12.5, and 19 knots) for each run. The three landfall points were selected so that the first landfall point was located at a distance equal to the radius of the maximum winds, west of the mouth of the Colorado River Navigation Channel at the barrier islands; the second point was centered on the mouth of the Colorado River Navigation Channel at the barrier islands; and the third was located a distance equal to the radius of the maximum winds east of the mouth of the Colorado River Navigation Channel, at the barrier islands. All storm tracks are straight. There are 81 combinations of these parameters, as stated before.

The PMH storm tracks used in the staff's independent analysis are shown in SER Figures 2.4S.5-1, 2.4S.5-2, and 2.4S.5-3. These figures are differentiated by track direction. Each figure shows seven track lines (three pairs offset to either side from a central track line by distances equal to different radii to maximum winds). In these figures, the central track is represented by the solid line, and the corresponding tracks to the west and the east are represented by matching broken lines. In each figure, several tracks use the same track line with different sets of track parameters (forward speed and radius to maximum winds). The total number of tracks represented in each figure is 27, with 9 along the central line and 3 along each of the other 6 lines.

A vertical datum offset is assigned in the SLOSH model to account for tides or other factors that cause the baseline sea level to be other than 0 m (0 ft) NGVD29. The staff cites this parameter to include the following:

- a 10 percent exceedance for astronomical tides (0.67 m [2.2 ft]) above the mean low water [MLW]) taken from RG 1.59 for Freeport, Texas (Table C.1)
- an initial rise of 0.73 m (2.4 ft) taken from RG 1.59 for Freeport, Texas (Table C.1)
- a 100-year sea-level rise of 0.44 m (1.43 ft) taken from the NOAA tide gauge at Freeport, Texas (NOAA, 2009)

Therefore, the staff used a datum offset of 1.84 m (6.03 ft) above the MLW when accounting for 100 years of sea-level rise and a datum offset of 1.4 m (4.6 ft) for present-day conditions. The MLW is 0.3 m (1.1 ft) below NGVD29 at the Freeport, Texas, tide gauge location. The vertical datum for the SLOSH is NGVD29 and therefore the staff used vertical offsets of 1.5 m (4.93 ft) (when accounting for sea-level rise) and 1.1 m (3.5 ft) for present-day conditions. The 81 PMH storm tracks were simulated for both of these cases in the staff's independent analysis.

The SLOSH simulations indicated that the maximum storm surge water surface elevation near the STP Units 3 and 4 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 toward the STP Units 3 and 4 site. The staff prepared a map of maximum water surface elevation on the SLOSH computational grid from these 81 PMH storm-track simulations for the two sea-level rise scenarios (Figure 2.4S.5-5). As expected, the staff's analysis obtained higher water surface elevations when the long-term sea-level rise was included before initiating the SLOSH simulations.

Because hurricanes rotate counter clockwise in the northern hemisphere, the highest surges are expected on the east side of the hurricane eye due to the fastest onshore wind being toward the right of the eye. Also, topographic highs provide some protection to land areas downwind from them and conversely lead to higher surge level to land areas upwind of them. Storms with larger forward speeds generate faster responses in surge, leaving less time for the surge to dissipate over and around the surrounding terrain. Considering these factors, the site would be most vulnerable to flooding when the eye of the hurricane passes quickly to the west of the site on the leading edge of the storm. These expected trends are borne out in the SLOSH results.

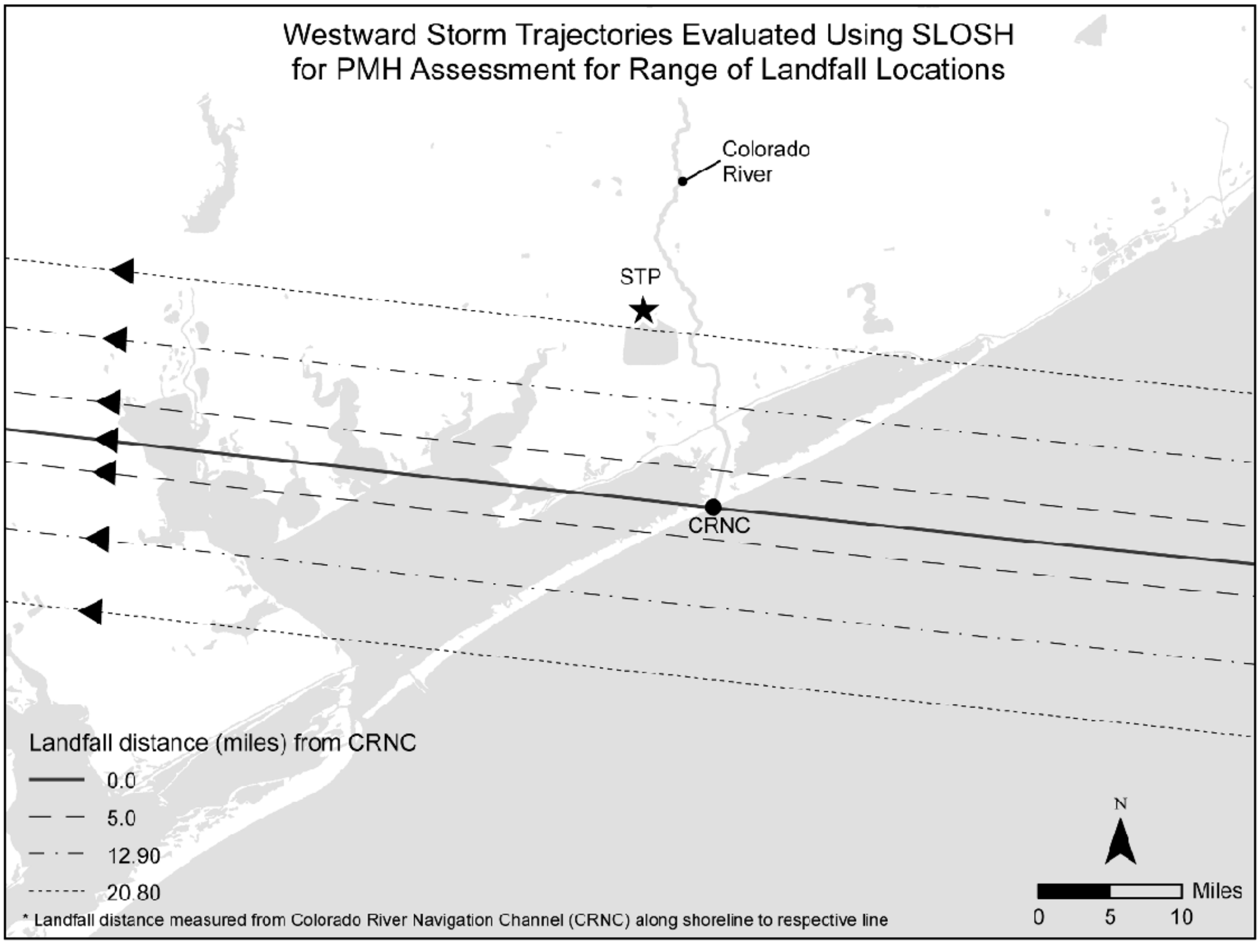

Figure 2.4S.5-1. Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

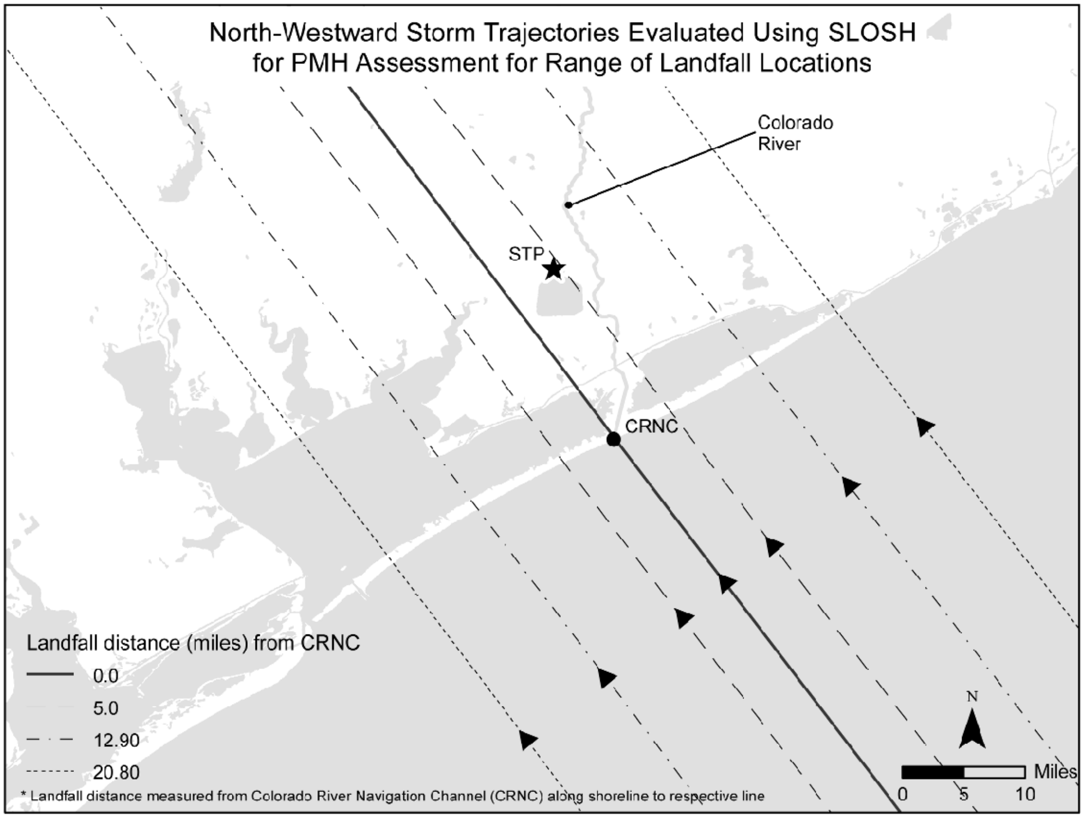

Figure 2.4S.5-2. North-Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

The version of the SLOSH model used by the staff has a limitation in terms of retaining and reporting computed water surface elevations. The model truncates any water surface elevations higher than 11 m (36 ft) NGVD29 and reports the values in those grid cells as a code, which means that in any grid cell that had a value set equal to this code, the storm surge water surface elevation exceeded 11 m (36 ft) NGVD29. Because the actual values of the storm surge water surface elevation are not retained for these grid cells, the only information available at these grid cells is that the maximum water surface elevation on the grid cells exceeded 11 m (36 ft) NGVD29. The staff's simulations resulted in the STP site being inundated during the most severe of the 81 PMH scenarios simulated and the storm surge water surface elevation on the grid cell where the STP Units 3 and 4 site is located exceeded 11 m (36 ft) NGVD29. Based on values of storm surge water surface elevations at surrounding grid cells, the staff estimated that the storm surge water surface elevation at the grid cell where the STP Units 3 and 4 site is located would probably be between 11.3 and 11.6 m (37 and 38 ft) NGVD29.

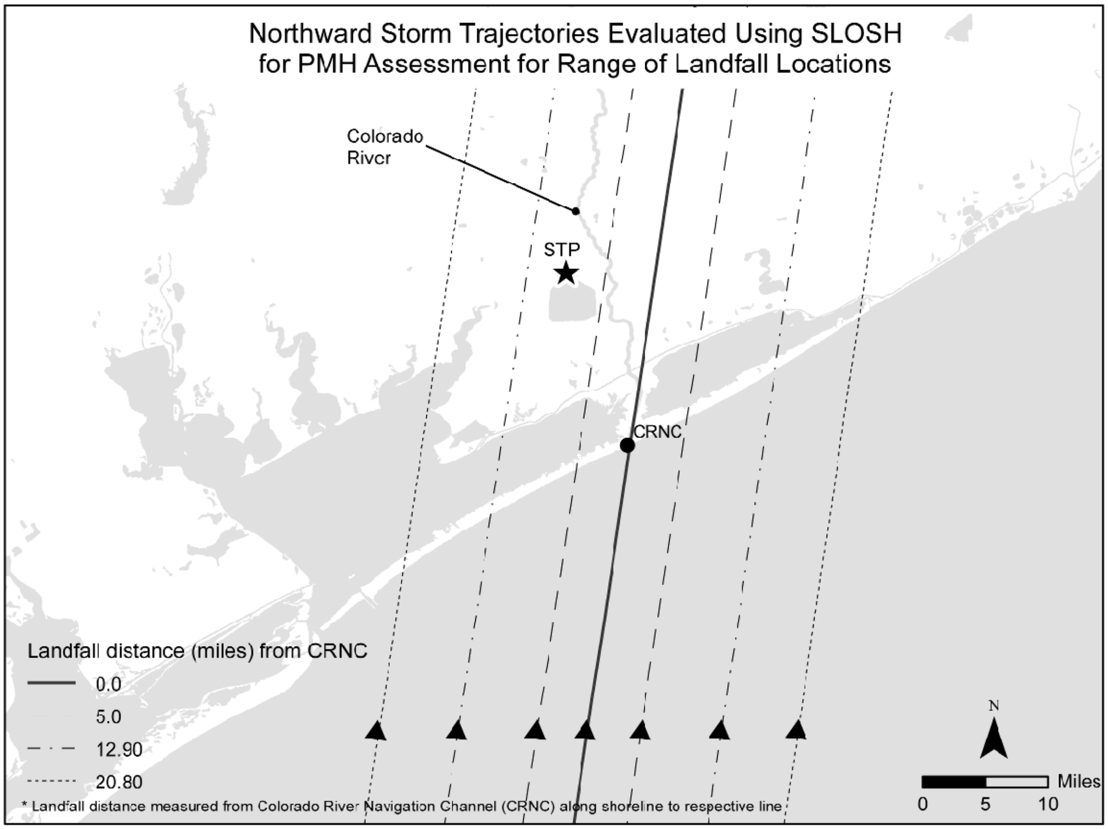

Figure 2.4S.5-3. Northward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations

Applicant ADCIRC Analysis

The staff reviewed the applicant's responses to RAI 02.04.05-11, parts (1) through (7). The staff's independent review found that the USACE ADCIRC model has a long history of development, verification, and validation (Luettich and Westerink, 1992; Luettich et al., 1992; Westerink et al., 1992; Blain et al., 1994; Grenier et al., 1994; Westerink et al., 1994; Luettich et al., 1998; Gica et al., 2001; Dietsche et al., 2007; Demirbilek et al., 2008; Westerink et al. 2008; Funakoshi et al., 2009; Bunya et al., 2010). The staff therefore determined that ADCIRC is an appropriate model for simulating storm surges from hurricane events. FEMA (2010) is currently using the ADCIRC model for flood insurance studies in coastal areas of the Atlantic Ocean and Gulf of Mexico.

The staff also reviewed the characteristics of bathymetry and near-shore topographic data used to represent the computational domain in the ADCIRC. The staff determined that the ADCIRC bathymetric and topographic data used by the applicant and contained in the TX2008 model are significantly more detailed than those used in the NRC SLOSH computational basins. The more detailed ADCIRC data resolve surface features with greater detail and accuracy. Another advantage of the ADCIRC model and computational grid is the ability to include topographic features at scales smaller than the grid resolution. These features allow the hydrodynamics to be simulated with much greater fidelity in the near-shore areas, because the hurricane storm surge interacts in complex ways with coastal features such as bays, estuaries, and rivers; and with buildings, roads, and levees. Two of the features that the ADCIRC computational grid resolves with greater vertical accuracy than in the SLOSH computational basin for the Matagorda Bay area are the City of Matagorda levee and the dredge piles along the lower Colorado River. In particular, the City of Matagorda levee lies directly in the path of a hurricane storm surge as it advances from the open waters of the Gulf of Mexico toward the STP site. The staff concluded that these features of the ADCIRC bathymetric and near-shore topographic data provide more detailed site-specific information for storm surge simulation at the STP site compared to the SLOSH model.

The staff also reviewed the applicant's statement provided in RAI response 02.04.05-11 that NWS 23 ranges of PMH parameters near the STP site include a radius to maximum winds of 9.7 to 33.5 km (6.0 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of 11.1 to 35.1 km/hr (6.9 to 21.8 mph). As described above, the staff independently obtained NWS 23 PMH parameter ranges including a radius to maximum winds of 9.7 to 33.5 km (6 to 20.8 mi), an approach direction of 97.5 to 190 degrees clockwise from the north, and a forward speed of the storm of 3.1 to 9.8 m/s or 11.1 to 35.2 km/hr (6 to 19 knots). The staff therefore concluded that the applicant has appropriately selected the PMH parameters from NWS 23.

The staff also reviewed the applicant's PMH scenarios provided in RAI response 02.04.05-11. The staff determined that the applicant-identified PMH scenario that would result in the largest storm surge at the STP site is consistent with the staff's independent SLOSH simulations described above. Based on these NRC SLOSH results, the applicant chose to simulate seven PMH scenarios in the ADCIRC. The staff determined that these seven scenarios are reasonably plausible for the STP site because they are consistent with the recommendations of NWS 23. The staff also determined that a larger, faster-moving hurricane produces a larger surge. Because the applicant uses a radius to maximum winds that is slightly more conservative than the one identified by the staff (38.6 km [24 mi] compared to staff's value of 33.5 km [21 mi]), and uses the same values for forward speed and central pressure difference, the staff concluded that the applicant has appropriately selected a conservative PMH scenario to simulate using the ADCIRC.

The above discussion provides the basis for the staff's determination that the applicant has selected conservative PMH scenarios for estimating the PMSS at the STP site. The staff also determined, as described above, that the applicant has selected an appropriate model supported by site-specific information. Therefore, the staff concluded that the applicant's ADCIRC simulations for determining the PMSS at the STP site are adequate and RAI 02.04.05-11 is considered closed.

NRC Staff and USACE ADCIRC Analysis

In 2009, in order to specify acceptable methods for estimating design-basis floods that reflect changes in state-of-the-art flood estimates since 1977—especially for regions susceptible to severe storm events—the NRC and the Army Corps of Engineers (USACE) conducted a project intended to provide the technical basis for estimating probable maximum floods due to the storm surge from extreme storm events, particularly along the U.S. southern coast, to evaluate flood protection for nuclear power plants.

As a result of the damage caused by the 2005 hurricane season (e.g., Hurricane Katrina), USACE created the Interagency Performance Evaluation Task Force (IPET). The IPET includes a distinguished group of government, academic, and private sector scientists and engineers and applies some of the most sophisticated capabilities available in civil engineering to understand what happened during Katrina and why. The purpose of the IPET was not only to acquire new knowledge, but to improve engineering practices and policies. In addition, the Congress of the United States authorized the USACE to initiate two important and comprehensive planning efforts that address the impacts caused by the 2005 storms and that would make the region more resilient and less susceptible to such profound harm from these disasters. One action plan included the Mississippi Coastal Improvements Program (MsCIP) which applied and further developed the technical approach and tools for estimating storm surge flood levels and waves established under the IPET. The USACE studies, tools, and approaches were extensively reviewed. Peer reviews were conducted by the distinguished External Review Panel (ERP) of the American Society of Civil Engineers and the National Academy of Sciences. The NRC/USACE project applied these tools and approaches to the South Texas Project, Levy County, and Turkey Point new reactor applications for the "Estimation of Very-Low Probability Hurricane Storm Surges for Design and Licensing of Nuclear Power Plants in Coastal Areas" (USACE ERDC, 2011).

The USACE hurricane modeling system used for the STP storm surge analysis combines various wind models, the WAM offshore and STWAVE nearshore wave models, and the ADCIRC basin-to-channel-scale unstructured grid circulation model. This is a well-validated modeling system that is applied to Corps projects. In addition, several FEMA regional offices have used it for flood mapping.

USACE and other agencies extensively use the Joint Probability Method (JPM) to determine hurricane storm parameters (synthetic storms) and to conduct storm hazard analyses (Resio and Irish, 2008; USACE ERDC, 2011). For synthetic storms, the TC96 Planetary Boundary Layer (PBL) model (Thompson and Cardone, 1996) is applied to construct snapshots of wind and atmospheric pressure fields every 15 minutes for driving the surge and wave models. Storms are defined by track- and time-varying wind field parameters. For each storm, a unique set of input conditions is defined. The data file includes the track position in space and time, the forward speed and direction, the central pressure, the pressure scale radius (which is related to the radius to maximum winds), a rotation angle, and a pressure profile peakedness parameter termed the Holland B factor (Holland, 1980). The wind and pressure field is generated and positioned on a fixed longitude/latitude grid system covering the Gulf of Mexico. Based on the location of the storm center, these snapshots describe the temporal and spatial evolution of a hurricane. The final wind and pressure fields resulting from the TC96 are targeted on a grid domain. The temporal variation in these fields is typically set to 1800 seconds (30-minute average wind). All wind fields are marine-exposure (no effective roughness variations for land/sea changes) and are generated at a 10-m (33-ft) elevation. The effect of ground cover on winds as the hurricane makes landfall is accounted for within the ADCIRC storm surge model.

Imposing the wind and atmospheric pressure fields, the depth-integrated circulation model ADCIRC (Luettich et al., 1992; Westerink et al., 1994; Luettich and Westerink, 2004) is run to replicate tide-induced and storm-surge water levels and currents. Parallel to the initial ADCIRC runs, the large-domain, discrete, time-dependent spectral wave model WAM (Komen et al., 1994) is run to calculate directional wave spectra that serve as boundary conditions for the local-domain, near-coast STWAVE wave model (Smith et al., 2001; Smith, 2007). The WAM generates the offshore wave field and directional wave spectra. The model solves the action-balance equation for the spatial and temporal variations of wave action in frequency and direction over a fixed longitude-latitude geospatial grid. The STWAVE model simulates nearshore wave transformation and generation. Using initial water levels from the ADCIRC, winds that include the effects of sheltering due to land boundaries and reductions due to land roughness, and spectral boundary conditions from the large-domain wave model, STWAVE is run to produce wave fields and to estimate radiation stress fields. The radiation stress fields are added to the estimated wind stresses and are then applied as forcing in the ADCIRC model, which estimates the water level across the entire grid at each time step.

Many coastal landscapes are characterized by complex bathymetry and topography. Natural features such as barrier islands, bays, inlets, marshes, lakes, and rivers—as well as man-made features such as levees, roadways, railways, navigation channels, gates, and seawalls—all influence surge and wave propagation. The surge and waves are not only influenced by the elevation of the landscape features, but also by land cover such as vegetation or buildings. The ADCIRC, TC96 PBL, and WAM model domains accurately capture basin-to-basin and shelf-to-basin physics, which is important in estimating high water levels that often occur well in advance of a hurricane's landfall.

In the NRC/USACE STP analysis, the ADCIRC mesh contains over 2.3 million nodes with nodal spacing reaching as low as approximately 40 m in the most highly refined areas. As demonstrated in the applicant's STP ADCIRC analysis, increased resolution across the coastal floodplain allows features such as inlets, rivers, navigation channels, levee systems, and local topography/bathymetry to be properly represented (Westerink et al., 1994). Levees and roadways are barriers to flood propagation that are generally below the defined grid scale. The ADCIRC defines these structures as sub-grid-scale parameterized weirs with a specified height (Westerink et al., 2001) within the domain. In addition, wave-breaking zones are resolved to ensure that the grid scales of the surge and nearshore wave models are consistent. The nearshore wave forcing function is properly incorporated by adding resolution where significant gradients in the wave radiation stresses exist (IPET, 2007; Bunya et al., 2009).

For a detailed, site-specific storm surge analysis, very extreme event storms are used that cover the range well beyond the annual exceedance probability of 10^{-6} (10^{-7} to 10^{-12}) (NRC, 1986). Two types of tracks span the range of physically realistic major storms approaching this site, storms that form in the Bay of Campeche to the south of the site and storms that enter the Gulf of Mexico between Cuba and the Yucatan (Figure 2.4S.5-4). A suite of 20 storms was developed and simulated with a coupled system of wind, wave, and coastal circulation models.

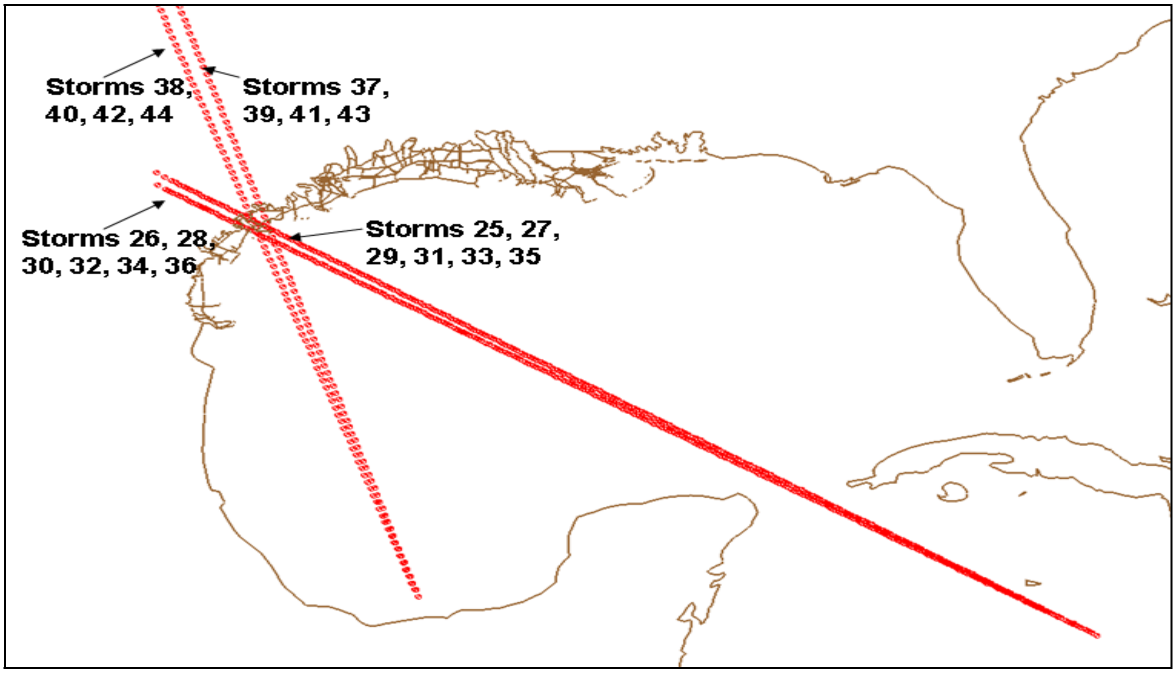

Figure 2.4S.5-4. Storm tracks developed with the maximum wind speeds over the point of interest

The Maximum Possible Intensity (MPI) of a hurricane was postulated as an upper limit for extreme tropical cyclone intensities at least since the late 1970s (for examples, see World Meteorological Organization, 1976 and Mooley, 1980). More recently, Emanuel (1986, 1991) and Holland (1997) formulated theoretical models for estimating the maximum tropical cyclone intensity. The central pressures used in the analysis were 880 mb (26.0 in. Hg), which is the lowest ever recorded for the Atlantic; and 870 mb (25.7 in. Hg), which is the lowest ever recorded worldwide. The radius of the maximum winds was 56 km to 83 km (30 nmi to 45 nmi). Note that by restricting the storm tracks to the paths shown in Figure 2.4S.5-4, the exceedance probability range could actually be lowered by one order of magnitude (i.e., to range from 10^{-8} to 10^{-13}).

As in the case of the applicant's ADCIRC simulations, a sea level rise of 0.59 m (1.93 ft) NAVD88, an initial rise of 0.79 m (2.6 ft) NAVD88, and the 10 percent exceedance high tide of 0.67 m (2.2 ft) NAVD88 were added to the ADCIRC still-water level calculations that included a wind wave and wave setup (STWAVE/WAM). There was no adjustment equal to the difference between the 10 percent exceedance high tide level and mean tide level, thus adding additional conservatism.

Table 2.4.5-4 contains the USACE Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory ADCIRC simulations adjusted for the STP site-specific storm surge characteristics. Twelve of the twenty storms produced a surge at the site. The ocean site characteristics were calculated in accordance with NRC guidance (RG 1.59 and NUREG–0800). In Table 2.4S.5-4, the PMSS for all 10^{-7} exceedance probability storms is 12.13 m (39.8 ft) NAVD88. The flooding level for the main cooling reservoir breach scenario in Section 2.4.4 is 12.19 m (40 ft) NAVD88. This table also shows the NRC SLOSH, the applicant's ADCIRC, and comparable USACE ADCIRC simulations for storms with similar meteorological parameters.

The NRC SLOSH and USACE ADCIRC have similar results, with a PMSS of 12.07 m (39.6 ft) NAVD88.

As previously mentioned, the staff determined that the ADCIRC bathymetric and topographic data used by the applicant and contained in the TX2008 model are significantly more detailed than those used in the NRC and USACE model computational basins. Thus, the difference between the NRC SLOSH/USACE ADCIRC and the applicant's ADCIRC analyses most likely reflects the presence of the two topographic features (the City of Matagorda levee and the dredge pile) in the applicant's Texas Grid version 13 that are not represented in the NRC SLOSH and USACE ADCIRC 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 applicant's ADCIRC storm surge from the Gulf toward the site.

Table 2.4S.5-4. USACE STP ADCIRC PMSS

PMSS (m/ft)	Surge (m/ft)	Wind (km/hr/mph)	P (mb/ in. Hg)	Rp (km/nmi)	Vf (km/ hr/mph)	DeltaP (mb/in. Hg)	Exceedance Probability
12.37/40.6	9.42/30.9	220/137	870/25.7	56-78/ 30-42	10/6	141/4.16	10^{-8}
12.22/40.1	9.27/30.4	230/143	870/25.7	56-78/ 30-42	21/13	141/4.16	10^{-8}
12.13/39.8	9.17/30.1	227/141	870/25.7	56-78/ 30-42	21/13	141/4.16	10^{-8}
12.13/39.8	9.17/30.1	216/134	880/26.0	56-78/ 30-42	10/6	131/3.87	10^{-7}
12.07/39.6	9.11/29.9	225/140	880/26.0	56-78/ 30-42	21/13	131/3.87	10^{-7}
12.01/39.4	9.05/29.7	214/133	870/25.7	83-117/ 45-63	10/6	141/4.16	10^{-12}
12.01/39.4	9.05/29.7	222/138	870/25.7	83-117/ 45-63	21/13	141/4.16	10^{-12}
11.95/39.2	8.99/29.5	240/149	870/25.7	56-78/ 30-42	40/25	141/4.16	10^{-8}
11.95/39.2	8.99/29.5	222/138	880/26.0	56-78/ 30-42	40/13	131/3.87	10^{-7}
11.86/38.9	8.90/29.2	219/136	870/25.7	83-117/ 45-63	40/13	141/4.16	10^{-12}
11.86/38.9	8.90/29.2	216/134	880/26.0	83-117/ 45-63	40/13	131/3.87	10^{-11}
11.58/38.0	8.60/28.2	209/130	880/26.0	83-117/ 45-63	10/6	131/3.87	10^{-11}

PMSS=probable maximum storm surge; mb=millibar; in. Hg=inches of mercury; m=meter; ft=foot; km/hr= kilometer per hour; mph=mile per hour; nmi=nautical mile

Table 2.4S.5-5. NRC SLOSH/Applicant ADCIRC vs USACE STP ADCIRC PMSS

PMSS (m/ft)	Surge (m/ft)	Wind (km/hr /mph)	P (mb/ in. Hg)	Rp (nmi)	Vf (mph)	DeltaP (mb/in. Hg)	Probability of Recurrence
STP ADCIRC							
9.13/29.95	------	296/184	887/26.2	39/21	37/23	133/3.93	------
NRC SLOSH							
12.07/39.6	------	241/149.7	887/26.2	39/21	35/22	133/3.93	------
USACE STP ADCIRC							
12.13/39.8	9.17/30.1	216/134	880/26.0	56-78/ 30-42	10/6	131/3.87	10^{-7}
12.07/39.6	9.11/29.9	225/140	880/26.0	56-78/ 30-42	21/13	131/3.87	10^{-7}
11.95/39.2	8.99/29.5	222/138	880/26.0	56-78/ 30-42	21/13	131/3.87	10^{-7}
11.86/38.9	8.90/29.2	216/134	880/26.0	83-117/ 45-63	21/13	131/3.87	10^{-11}
11.58/38.0	8.60/28.2	209/130	880/26.0	83-117/ 45-63	10/6	131/3.87	10^{-11}
PMSS=probable maximum storm surge; mb=millibar; in. Hg=inches of mercury; m=meter; ft=foot; km/hr= kilometer per hour; mph=mile per hour; nmi=nautical mile							

2.4S.5.4.3 Wave Action

Information Submitted by Applicant

The applicant determines that wave action coupled with the probable maximum surge is not the controlling wave scenario. The applicant assesses wave action coupled with flooding described in FSAR Section 2.4S.4 to be more conservative.

The applicant uses the USACE ADCIRC model to perform PMSS estimation. The applicant's ADCIRC model is tightly coupled with the SWAN model that computes wind waves within the ADCIRC-SWAN runs. The applicant states that the maximum PMSS water surface elevation of 8.9 m (29.3 ft) MSL includes wind-wave effects.

NRC Staff's Technical Evaluation

The staff conservatively estimated the maximum PMH storm surge water surface elevation to be between 11.3 and 11.6 m (37 and 38 ft) NGVD29 near the STP Units 3 and 4 site. The water depth near the site would be approximately 0.9 to 1.2 m (3 to 4 ft) at this location. For this shallow water depth, the PMH wind speeds, and unlimited fetch, the staff estimated the wind-wave amplitude to be 0.27 to 0.36 m (0.9 to 1.2 ft) following the methods in the Coastal

Engineering Manual (USACE, 2008). The wave runup is a function of the depth of the water and the ground slope over which the wave passes. The ground slope is not precisely known, so a range of reasonable values was used. As the ground steepens, wave runup becomes higher. Based on the conservative assumption of an armored shore, the staff used a steepest slope of 10 percent. The staff determined the corresponding conservative wave runup to be approximately 0.20 m (0.65 ft). Therefore, an evaluation of wave action shows that it adds 0.47 to 0.56 m (1.55 to 1.85 ft) to the peak level of inundation estimated by the SLOSH simulations.

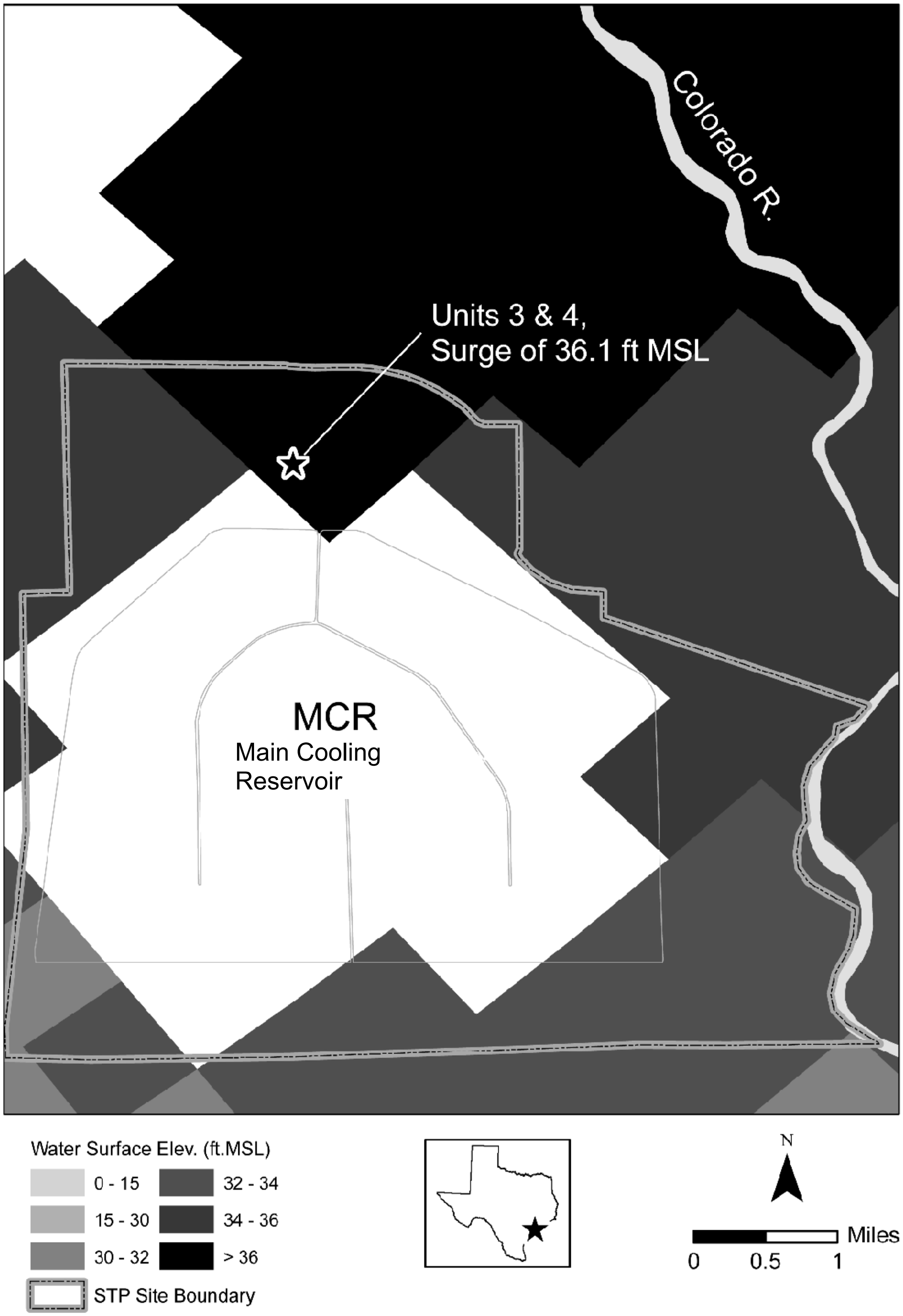

Figure 2.4S.5-5. NRC Staff-Estimated PMSS Water Surface Elevations at the STP Site

Therefore, the staff estimated the maximum PMH storm surge water surface elevation to be between approximately 11.6 to 12.1 m (38.0 to 39.6 ft) NGVD29, including the effects of wind waves at the STP Units 3 and 4 site (see SER Table 2.4S.5-5).

To compare the relative severity of the PMH parameters estimated from NWS 23 (based on hurricane data from 1851–1978), the staff compared these parameters to severe storm studies currently being carried out (Resio 2009; Vickery, 2009). The Resio and Vickery storms are derived from the NOAA hurricane database (HURDAT), which is the official record of tropical storms and hurricanes for the Atlantic Ocean, Gulf of Mexico, and Caribbean Sea. The hurricane data are from 1851 through 2011. The staff found that the PMH estimated by the NWS 23 method is smaller in size than those estimated near the STP site by Resio (2009), but it has greater wind speeds. On the other hand, the severe storms estimated by Vickery (2009) near the STP site are smaller in size than the PMH, but they have slightly greater wind speeds. The storm surges estimated by Resio (2009) inundate the STP site. However, the maximum stillwater surface elevations are less than those estimated by the staff's independent analysis described above. The Vickery (2009) simulations of storm surge at the STP site were also carried out using the SLOSH model, but they did not result in the inundation of the STP site.

Based on the above information, the staff concluded that the PMH estimated from the NWS 23 method is appropriate to estimate a reasonably conservative maximum storm surge water surface elevation at the STP site.

The staff determined that the applicant's site-specific PMSS maximum water surface elevation of 8.9 m (29.3 ft) MSL is reasonable and conservative. Although the applicant does not provide an estimate of the wind-wave runup (the wind-wave setup is accounted for in the ADCIRC simulations), the staff determined that the applicant's independent estimate of 0.20 m (0.65 ft) could be conservatively added to the applicant's PMSS stillwater and wind setup estimate, because the staff's estimate is derived from a more conservative PMSS scenario. Therefore, the staff concluded that the maximum PMSS water surface elevation at the STP Units 3 and 4 site accounting for the wind setup and runup would not exceed 9.1 m (30 ft) MSL and would be 0.6 to 1.2 m (2 to 4 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL. Because the PMSS maximum water surface elevation accounting for wind-wave effects is below the site grade and is exceeded by the maximum water surface elevation expected during the postulated main cooling reservoir embankment breach event, the staff concluded that further investigation of the PMSS at the STP site is not warranted.

2.4S.5.4.4 Resonance

Information Submitted by Applicant

The applicant identifies no scenario that will produce resonance effects.

NRC Staff's Technical Evaluation

The applicant states that there is no scenario that would produce resonance effects. FSAR Section 2.4S.8 analyzes PMH winds as a potential mechanism for the generation of resonant seiches in the main cooling reservoir (STPNOC, 2007). Consideration of the geometry and water depths of the main cooling reservoir allows for estimates of the necessary wind-wave frequency that could lead to a seiche; the differences between the PMH wind wave and the natural resonant frequency leads to the conclusion that there is no possibility of this postulated coupling.

The staff issued RAI 02.04.05-7 requesting the applicant to provide an assessment of seismically induced seiches in the main cooling reservoir. In a letter dated August 12, 2008 (ML082270381), the applicant's response states that there was no consideration of seiche effects in the main cooling reservoir from seismic forcing. The applicant considers the main cooling reservoir embankment failure as the bounding case for site flooding and a design-basis flood for the STP Units 3 and 4 site. The staff performed an independent assessment of seismic seiche in the main cooling reservoir. Section 2.4S.8 of this SER discusses the staff's independent assessment, including resonance in the main cooling reservoir.

2.4S.5.4.5 Protective Structures

Information Submitted by Applicant

The applicant assesses the flood-level estimate from the postulated main cooling reservoir embankment breach to be the controlling event related to safety-related facilities. This analysis is discussed in FSAR 2.4S.4.

NRC Staff's Technical Evaluation

The applicant considers the flood generated by a postulated failure of the main cooling reservoir embankment to be the controlling flood at the STP Units 3 and 4 site and therefore, the design basis flood for protecting all safety-related SSCs. The staff's independent assessment in this section indicated that the PMH storm surge would flood the STP Units 3 and 4 site. The staff also determined that the PMH storm surge would result in floodwaters surrounding the main cooling reservoir embankment. The staff postulated an induced failure of the main cooling reservoir embankment because of the sloshing and erosive action of floodwaters surrounding the main cooling reservoir during a PMH storm surge event, as described below.

The staff reviewed the applicant's responses regarding flood protection requirements. The staff concluded that there is one combined event scenario that the applicant did not address in the FSAR. The staff estimated the storm surge resulting from a PMH above. The staff concluded that although the PMH storm surge water surface elevation at the STP Units 3 and 4 site will not exceed the floodwater surface elevation resulting from the postulated breach of the main cooling reservoir north embankment, it may provide a trigger for the failure of the main cooling reservoir embankment. The outside surface of the main cooling reservoir embankment is lined with grass and is not protected by any riprap or armoring, which makes the embankment vulnerable to storm surge currents and erosion. However, the main cooling reservoir embankment was constructed using primarily clay soils from the site compacted to uniform densities across the entire embankment cross section. The construction adhered to stringent compaction control measures described in Section 2.5.6 of the STPEGS UFSAR (Units 1 and 2). In addition, the slope of the exterior face of the embankment was constructed at 3 horizontal to 1 vertical and is covered with topsoil and seeded to protect against erosion. The surface of the interior face of the embankment is protected by a layer of soil cement that is 30 in. thick (STPEGS UFSAR, Subsection 2.4.4.1.1.3).

The main cooling reservoir embankment is equipped with a seepage-control system consisting of a sand drain blanket, relief wells and a compacted impervious clay embankment around the reservoir to protect the toe of the embankment by lowering the seepage level. However, the seepage control system is lower than the system at the PMH surge level and will not be functioning during the surge inundation, which could trigger a main cooling reservoir embankment breach. Therefore, the staff postulated that it is possible for the main cooling reservoir embankment to fail while being under inundation during the PMH storm surge. As an example, the 2005 New Orleans flooding from Hurricane Katrina was caused by a combination of storm surge and levee failure, where the failure was caused by both seepage and overtopping (U.S. General Accounting Office [GAO], 2006). The staff therefore determined that for the STP site, a combination of the PMH storm surge and main cooling reservoir embankment failure needs to be investigated to estimate the maximum floodwater surface elevation at the STP Units 3 and 4 site.

Based on the applicant's FSAR the staff determined that the applicant has not shown that the ADCIRC model results account for the most conservative plausible PMH scenario. Furthermore, the description and result of these model applications are not included in the FSAR. Therefore the staff issued RAI 02.04.05-11, which was tracked as Open Item 2.4.5-1. The applicant's ADCIRC PMSS maximum water surface elevation accounting for wind-wave effects is below the site grade (8.3 m [29 ft] versus 10.4 m [34 ft] MSL site grade) and is exceeded by the maximum water surface elevation expected during the postulated main cooling reservoir embankment breach event (12.19 m [40 ft] MSL). Similarly, the NRC SLOSH PMSS (12.07 m [39.6 ft] MSL) and the USACE ADCIRC PMSS (12.13 m [39.8 ft] MSL) levels are also exceeded by the main cooling reservoir breach event, thus resulting in a main cooling reservoir freeboard of 8 to 11 m (25 to 36 ft). Therefore, no overtopping from an external storm surge event is expected.

In the response to RAI 02.04.050-11 dated November 22, 2010 (ML111510810), the applicant provides three different methods to estimate the current velocities along the external face of the main cooling reservoir northern embankment for the NRC SLOSH-modeled scenario. The values were 3.5 m/s (11.6 ft/s), 0.9 m/s (3.1 ft/s), and 1.9 to 4.0 m/s (6.2 to 13.2 ft/s) with the flow along the embankment occurring for up to 80 minutes. For this duration, Hewlett et al. (1987) state that depending on the quality of the grass cover, grass-lined channels can sustain velocities of 2.7 to 4.3 m/s (9 to 14 ft/s) (see Figure 9 in Hewlett et al.1987). The predicted velocities are comparable and suggest that the grass cover would be able to withstand this level of a hydraulic attack. Even if the grass cover were damaged within this time frame, the clay content of the underlying zone B materials (clay with a liquid limit ≥ 30) suggests that these materials would have at least a moderate resistance to erosion. Thus, it seems very unlikely that subsequent damage to the underlying embankment materials could be sufficient in this time period to lead to a main cooling reservoir embankment breach. The maximum mean current velocities that are considered to be safe against erosion are 1.2 to 1.5 m/s (4 to 5 ft/s) for stiff clay soil and ordinary gravel (Fortier and Scobey, 1926; Connecticut Council for Soil and Water Conservation, 1985). In addition, for a material of a given plasticity index, the permissible shear stress increases nearly ten-fold when the material is properly compacted (New Orleans Systems Hurricane Katrina, 2006). The staff calculated maximum current velocities of 1.3 m/s (4.4 ft/s) and 1.2 m/s (4 ft/s) for the NRC SLOSH and USACE ADCIRC storm surges, respectively, which fall within the maximum mean current velocities of 1.2 to 1.5 m/s (4 to 5 ft/s) that are considered to be safe against erosion for a stiff clay soil. Finally, because the applicant's ADCIRC PMSS is below site grade (10.4 m [34 ft]) and is equal to the main cooling reservoir north embankment grade level (8.8 m [29 ft]), the main cooling reservoir embankment is safe against erosion.

Based on the above data, the staff concluded that no further investigation of the PMSS at the STP site is warranted. The applicant also has new and updated information that will be included in a future revision of the FSAR. Therefore, Open Item 2.4.5-1 is closed. The staff confirmed that FSAR Revision 7 includes the proposed FSAR text changes.

2.4S.5.5 Post Combined License Activities

There are no post COL activities related to this subsection.

2.4S.5.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to estimating the flood levels caused by a hurricane storm surge from the Gulf of Mexico, and no outstanding information is required to be addressed in the COL FSAR related to this section. The staff determined that the applicant's site-specific PMSS maximum water surface elevation of 8.9 m (29.3 ft) MSL is reasonable and conservative.

NWS 23 covers the period of 1871 to 1978. The applicant adequately addresses the issue of how conservative the PMH parameters are in light of more recent hurricanes that have occurred since the publication of the NOAA NWS 23 report. NRC staff independently determined that 54 hurricanes have impacted Texas between 1851 and 2008, with 18.5 percent occurring outside of the NWS 23 reporting period. All Category 4 hurricanes impacting Texas occurred within the NWS 23 reporting period, and no hurricane greater than a Category 4 has ever made landfall in Texas. For the United States, only 17 percent of all hurricanes that have impacted the country occurred after the NWS 23 reporting period. Among the 12 most intense hurricanes to hit this country, only 3 occurred outside of the NWS 23 reporting period.

The applicant's wind speed of 296 km/hr (184 mph), with no decay at landfall, is 6.4 km/h (4 mph) greater than the highest recorded hurricane speed in Texas (290 km/hr [180 mph] in 1970 at Port Aransas) and exceeds what is currently classified as a Category 5 hurricane. In addition, because the applicant's ADCIRC analysis uses a radius to maximum winds that is slightly more conservative than the one identified by the staff (38.6 km [24 mi] compared to the staff's value of 33.5 km [21 mi]), and because the applicant uses the same values as the staff's SLOSH analysis for forward speed and central pressure differences, the staff concluded that the applicant has appropriately selected a conservative PMH scenario to simulate using the ADCIRC model.

Finally, the applicant's ADCIRC bathymetric and nearshore topographic data provide more detailed site-specific information for a storm surge simulation at the STP site compared to the staff SLOSH and the USACE ADCIRC models. There are two features that the applicant's ADCIRC computational grid resolves with a greater vertical accuracy compared to the NRC SLOSH and the USACE ADCIRC computational basin for the Matagorda Bay area: the City of Matagorda levee and the dredge piles along the lower Colorado River. In particular, the City of Matagorda levee lies directly in the path of a hurricane storm surge as it advances from the open waters of the Gulf of Mexico toward the STP site, which results in the applicant's lower PMSS (8.9 m [29.3 ft] MSL) compared to the staff SLOSH PMSS (12.07 m [39.6 ft] MSL) and the USACE ADCIRC PMSS (12.13 m [39.8 ft] MSL). Because the applicant's ADCIRC PMSS is below the site grade (10.36 m [34 ft] MSL) and equal to the main cooling reservoir north embankment grade level (8.84 m [29 ft] MSL), the main cooling reservoir embankment is safe against erosion. Note that a storm surge of 29 ft equals or exceeds the Hurricane Katrina (2005) storm surge, which is currently the highest recorded storm surge in U.S. history.

As set forth above, the applicant presents and substantiates information to establish the site description. The staff reviewed the applicant's information and for the reasons stated above, the staff concluded that, as documented in Section 2.4S.5 of this SER, the applicant has provided sufficient detail about the site description to allow the staff to evaluate whether the applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site. The information addressing COL Information Items 2.4 and 3.5 is therefore accurate and acceptable.

2.4S.5.7 References

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2.4S.6 Probable Maximum Tsunami

2.4S.6.1 Introduction

This section of the FSAR addresses the hydrological design basis developed to ensure that any potential tsunami hazards to the SSCs important to safety are considered in the plant's design.

This SER section presents the staff's review of the flood levels caused by postulated tsunami scenarios. The specific areas of the review include the description of the PMT, historical tsunami records, source generator characteristics, tsunami analyses, tsunami water levels, hydrography and harbor or breakwater influences on a tsunami, and the effects on safety-related facilities.

2.4S.6.2 Summary of Application

In Section 2.4S.6, the applicant provides site-specific information about potential tsunami effects on the site. In addition, in FSAR Section 2.4S.4, the applicant provides site-specific information to address COL License Information Items 2.14 and 3.5:

COL License Information Items

- COL License Information Item 2.14 Floods
- COL License Information Item 3.5 Flood Elevation

COL License Information Item 2.14 requires COL applicants to provide site-specific information related to historical flooding and the potential flooding at the plant site including flood history, flood design considerations, and effects of local intense precipitation. This information is provided below.

In FSAR Section 2.4S.6, the applicant evaluates several different tsunami sources from the published scientific literature to establish the 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 applicant.

After reviewing published tsunami catalogs, databases, and historical accounts, the applicant identifies the following three historical tsunami events for 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 (not a tsunami event in the Gulf of Mexico)

The applicant examines 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 applicant discusses 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 applicant cites recent studies to discount the La Palma, Canary Islands transoceanic tsunami scenario published by Ward and Day (2001). For landslide-generated tsunamis, the applicant discounts the East Breaks landslide tsunami scenario published by Trabant et al., (2001) as highly unlikely.

To determine the maximum tsunami water levels, the applicant uses an estimate of the tsunami in the Gulf of Mexico from a near-field submarine landslide near the East Break slump and then applies (1) a runup amplification factor, (2) 10 percent exceedance of an astronomical high tide according to RG 1.59, Revision 2, and (3) a sea level rise from global climate change in the next century. The applicant determines the maximum water level for the PMT at 11.5 feet above MSL.

Therefore, the applicant concludes that the flood elevation at STP Units 3 and 4 due to the postulated PMT event will not be the controlling design-based flood elevation for STP Units 3 and 4 because it is below the plant grade, and there will be no onsite effects from tsunami-breaking waves or resonance or onsite tsunami waves on safety-related facilities.

2.4S.6.3 Regulatory Basis

The relevant requirements of the Commission regulations for the PMT, and the associated acceptance criteria, are in Section 2.4.6 of NUREG-0800.

The regulatory requirements that establish the acceptance criteria for reviewing this section are as follows:

- 10 CFR Part 50, Appendix A, GDC 2, requires the COL applicants to consider the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 52.79(a)(1)(iii), requires the COL applicants to identify hydrologic site characteristics with appropriate consideration for the most severe of the natural phenomena that have been historically reported for the site and surrounding areas, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 100.20 specifies the factors to be considered when evaluating sites.
  10 CFR 100.20(c) specifies the requirements for considering the physical characteristics of a site (including seismology, meteorology, geology, and hydrology) to determine its acceptability to host a nuclear unit(s).

- 10 CFR 100.23(d)(3) sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.
  Section IV(c) of Appendix A to Part 100 specifies the required information for seismically induced floods and water waves, including distantly and locally generated tsunami runup and drawdown, local coastal topography that affects the tsunami runup and drawdown, geologic and seismic evidence for evaluating seismically induced floods and water waves, and probable slip characteristics of offshore or nearby lakes and rivers.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27 describes the applicable UHS capabilities.

- RG 1.59, as supplemented by the best current practices, provides guidance for developing the design flood bases.

2.4S.6.4 Technical Evaluation

NRC staff reviewed the information in Section 2.4S.6 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the PMT. The staff's technical review of this section included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information in FSAR Section 2.4S.6.

COL License Information Items

- COL License Information Item 2.14 Floods

- COL License Information Item 3.5 Flood Elevation

The staff reviewed the applicant's supplemental information on tsunami-generated floods. The staff's review of the application is summarized below:

2.4S.6.4.1 Probable Maximum Tsunami

Information Submitted by Applicant

The applicant evaluates several different tsunami sources from the published scientific literature to establish the PMT. Approximate tsunami wave heights are indicated for four seismogenic sources located in the Caribbean and the Gulf of Mexico and for the 1755 Lisbon earthquake, which was located in the Atlantic Ocean. In the FSAR, the applicant states that the wave height estimate for the East Breaks submarine landslide is highly unlikely. However, the applicant revises the potential for tsunamis from the East Breaks landslide in the response to RAI 02.04.06-1 dated December 4, 2008 ML083460084) and in the FSAR.

RAI 02.04.06-1 requests the following information:

> [Item 1] Provide a tsunami modeling analysis of the East Breaks landslide to clarify whether the 7.6-m (24.93-ft) offshore wave height indicated by Trabant et al., (2001) can be discounted.
>
> [Item 2] In addition, provide additional tsunami analyses of other regions in the Gulf of Mexico that are prone to landslides.

[Item 3] To independently validate whether a tsunami hazard exists for the proposed site, provide geologic methods and tsunami identification criteria used to justify the determination that no tsunami deposit was found at the site.

[Item 4] Provide excavation photos from Units 1 and 2.

[Item 5] Indicate if there are geologically conducive locations for the deposition and preservation of tsunami deposits at the STP site or nearby regions.

NRC Staff's Technical Evaluation

Resolution of the significant items [Items 1 and 2] of RAI 02.04.06-1 is discussed below.

[Item 1] East Breaks Landslide: The applicant's response to the RAI, dated December 4, 2008 (ML083460084), provides the geologic background and four possible source scenarios for landslide tsunamis in the East Breaks region. The geologic background for the East Breaks landslide is taken primarily from published literature and, in general, presents a reasonable summary. The applicant also provides the theoretical basis of the tsunami propagation used (Method of Splitting a Tsunami [MOST]) and its verification. However, the applicant does not thoroughly discuss the conservatism of input parameters. The applicant uses a large (but physically reasonable) bottom-roughness coefficient (i.e., 0.01 on page 4 of the response) that may not give the most conservative estimate of the water level. The generation phase of the applicant's simulations is based on a slump center-of-mass motion model, in which the time history of slide movement is specified only for the center of the mass of a slide with a prescribed geometry (e.g., Gaussian shape). This model contrasts with using the full-time varying displacement field for submarine mass failures as initial conditions for tsunami generation. The center-of-mass motion model may be adequate during the early stages of a post-failure slide movement but does not account for changes in deformation, as the landslide fully mobilizes down the slope. The staff determined that the response to RAI 02.04.06-1, Item 1 is acceptable.

[Item 2] Other Gulf of Mexico Landslides: The applicant provides a descriptive justification for why other Gulf of Mexico landslide provinces are not considered in establishing the PMT for the site. These provinces are the Mississippi Canyon, Florida Escarpment, and Campeche Escarpment (ten Brink et al., 2008). The applicant maintains that there is a significant diffusion and energy dissipation associated with landslides that are more distant than the East Breaks landslide. It is unclear whether the applicant performed an additional tsunami analysis for the more distant landslides to make this conjecture.

In the FSAR the applicant concluded that the more distant landslides in the Gulf of Mexico with propagation paths oblique to the site are not likely to have potential runup heights greater than those from the East Breaks Landslide. However, the applicant does not provide sufficient justification for dismissing the possibility that the Campeche Escarpment region may be a potential source region that determines the PMT water levels. To evaluate the potential tsunami effects of these submarine landslide sources, the staff performed an independent confirmatory analysis that estimated the PMT water levels.

<u>Confirmatory analysis and major findings</u>: A detailed description of the NRC staff's independent confirmatory analysis to determine the PMT at the STP site is in the sections that follow. In summary, the staff considered both far-field seismogenic and near-field (Gulf of Mexico)

landslide sources as potential generators for the PMT. An initial analysis indicates that submarine landslides broadside (i.e., directly across) from the site are the likely sources that determine the PMT (See SER Subsection 2.4S.6.4.3). This analysis includes the East Breaks landslide and potential landslides along the Campeche Escarpment. Each landslide source has a unique hydrodynamic behavior described below in Subsection 2.4S.6.4.5. Within the uncertainty of the tsunamigenic source data, either could be the PMT source.

Conclusion: The applicant's response to RAI 02.04.06-1 and the NRC staff's confirmatory analysis differ significantly in the descriptions of how to determine the PMT. However, the applicant's PMT water level estimate (3.5 m [11.5 ft] MSL) represents a near-shore/coastal location that is less than the staff's PMT water level estimate of 5 m (16.4 ft) MSL for an inland location closer to the STP site, taking into account the effect of an overland flow. Moreover, the PMT surge level estimates by both the applicant and the staff are far below the bounding main cooling reservoir breach water level of 12.2 m (40.0 ft) MSL or the plant grade of 10.36 m (34 ft) MSL. Thus, the staff concluded that the postulated PMT would not affect the proposed STP site. Therefore, RAI 02.04.06-1 is closed.

2.4S.6.4.2 Historical Tsunami Record

Information Submitted by Applicant

After reviewing published tsunami catalogs, databases (such as National Geodetic Data Center), and historical accounts, the applicant identifies three historical tsunami events for the STP site. These include (1) an October 11, 1918, seismogenic tsunami originating west of Puerto Rico; (2) a May 2, 1922, seismogenic tsunami originating near the Virgin Islands; and (3) seismic seiche waves originating from the March 27, 1964, Gulf of Alaska earthquake (not a tsunami event in the Gulf of Mexico).

NRC Staff's Technical Evaluation

NRC staff conducted a review of this historical record to confirm whether the three events listed by the applicant are the primary tsunamis and seismic seiches measured and observed along the Gulf Coast. An additional entry in the National Geodetic Data Center (NGDC) tsunami database for the Gulf of Mexico is an event that occurred at Grand Isle, Louisiana, on September 22, 1909. As indicated in the database, this event was likely caused by a hurricane, not by a tsunami. See Geist et al., (2009) for a discussion of other historic tsunamis.

The applicant does not address possible evidence for paleotsunami deposits in the FSAR Section 2.4S.6. For example, a deposit located north of the site in Falls County, Texas, near the Brazos River was originally interpreted by Bourgeois et al., (1988) as caused by a paleotsunami. The Brazos deposit is dated at or near the time of the Cretaceous-Tertiary boundary and is located at the paleo-shoreline for that time period. Since that time, the Gulf Coast shoreline has transgressed southward to its current geographic position. The common interpretation of this deposit is that owing to its date and the existence of impact ejecta, it was emplaced by a tsunami generated from a Chicxulub asteroid impact at the Brazos site. Bourgeois et al., (1988) suggested that a tsunami wave 50 to 100 m (164 to 328 ft) high was necessary to explain this deposit. It is not conceivable that the wave that created these deposits was generated by any landslide source that would be of relevance to the present day PMT determination. It is likely that a wave of the estimated height would be caused by a relatively nearby large impact event. Waves emanating from such a source would have the extreme wave heights and long periods needed to be able to propagate significant wave energy this far inland.

Conclusion: NRC staff examined primary references for historical observations and measurements of tsunami and seismic seiche waves occurring along the Gulf Coast. Except for the date of the 1918 hydrologic event and the source for the 1922 hydrologic event, the staff's assessment of the historical record is consistent with that of the applicant's. Additionally, the applicant does not consider the existence of a possible paleotsunami that occurred along the ancient Gulf Coast shoreline, currently located along the Brazos River in Falls County, Texas. The common interpretation of this deposit is that it was emplaced by a tsunami generated by the Chicxulub impact. It is unlikely, however, that the wave heights inferred from the deposit are relevant to a determination of the present day PMT. Therefore, the staff finds the applicant's analysis acceptable.

Source Generator Characteristics

Information Submitted by Applicant

In FSAR Subsection 2.4S.6.3, the applicant states that it examined published information to determine the source generator characteristics for several different types of tsunamis: seismogenic, volcanogenic, and landslide generated. For seismogenic tsunamis, the applicant discusses the propagation characteristics into the Gulf of Mexico for earthquakes located in the Caribbean and the Atlantic Ocean. For volcanogenic tsunamis, the applicant cites recent studies to discount the La Palma, Canary Islands transoceanic tsunami scenario. For landslide-generated tsunamis, the applicant discounts the East Breaks landslide tsunami scenario published by Trabant et al., (2001) as highly unlikely, though the applicant revisits this scenario in the response to RAI 02.04.06-1.

NRC Staff's Technical Evaluation

This section describes the tsunami sources used for the independent confirmatory analysis, including parameters associated with the maximum submarine landslides in the Gulf of Mexico. The end of this section includes a brief discussion of seismic seiches.

Potential tsunami sources that are likely to determine the PMT at the STP site are submarine landslides in the Gulf of Mexico. Subaerial landslides, volcanogenic sources, near-field intra-plate earthquakes and inter-plate earthquakes along the Caribbean plate boundary faults are unlikely to be the causative tsunami generator for the PMT at the STP site.

Subaerial Landslides: With regard to subaerial landslides, there are no major coastal cliffs near the site that would produce tsunami-like waves that exceed the amplitude of those generated by other sources.

Volcanogenic Sources: According to the Global Volcanism Program of the Smithsonian Institution (http://www.volcano.si.edu/), there are three general regions of volcanic activity that have the potential to generate localized wave activity in the Gulf of Mexico and the Caribbean Sea: (1) two Mexican volcanoes near the Gulf of Mexico coastline, (2) two volcanoes in the western Caribbean, and (3) volcanic activity along the Lesser Antilles island arc. Catastrophic failures associated with volcanoes along the eastern coasts of Mexico and Central America is either too far inland or too small in size to generate significant wave activity near the STP site.

Based on existing evidence, volcanoes along the Lesser Antilles or in the eastern Atlantic Ocean are too far away to generate significant wave activity in the Gulf of Mexico.

Intra-Plate Earthquakes: Because there are no tectonic plate boundaries in the Gulf of Mexico region, earthquakes *local* to the STP site occur in an intra-plate tectonic environment, thus limiting the maximum magnitude these earthquakes can attain ($M_{max} = 7.5$; see Petersen et al., 2008, for details of this analysis). Because the maximum slip, and consequently the maximum sea floor displacement, associated with an earthquake scale with its magnitude, the initial tsunami wave amplitude associated with an intra-plate earthquake would therefore be less than that used for local submarine landslides under conservative conditions, as described below in Subsection 2.4S.6.4.5.

Inter-Plate Earthquakes: In the far-field description of major plate boundary faults, Chapter 8 of ten Brink et al., (2008) estimates specific source parameters and offshore tsunami amplitudes of Caribbean inter-plate earthquakes. The tsunami propagation model in ten Brink et al., (2008) was refined during the NRC staff's confirmatory analysis for two of the principal faults (the northern South American Convergent Zone and the northern Caribbean Subduction Zone) using the Cornell Multigrid Coupled Tsunami Model (COMCOT) (See Subsection 2.4S.6.4.5 below).

Local Submarine Landslides: Submarine landslides in the Gulf of Mexico are considered a potential tsunami hazard for the STP site for two reasons: (1) some dated landslides in the Gulf of Mexico have post-glacial ages, suggesting that the triggering conditions for these landslides are still present; and (2) analyses of recent seismicity suggest the presence of small-scale energetic landslides in the Gulf of Mexico. NRC staff defined four geological provinces in the Gulf of Mexico that are likely to be the origin of submarine landslides that control the determination of the PMT: northwest of the Gulf of Mexico (immediately off the STP site), the Mississippi Canyon, the Florida Escarpment, and the Campeche Escarpment. The first is a mixed canyon/fan and salt province involving the failure of terrigenous and hemipelagic sediment; the second is a canyon/fan province; and the third and fourth are carbonate provinces formed from reef structures and characterized by steep slopes (i.e., escarpments).

Because the Mississippi Canyon and Florida Escarpment landslides are oblique to the STP site, the length of the continental shelf that the wave must travel over is much greater than that for the East Breaks landslide or for landslides along the Campeche Escarpment that are broadside from the STP site. This would result in much greater energy dissipation during propagation that is associated with tsunamis from the Mississippi Canyon and Florida Escarpment source regions. The characteristics and the parameters that define the maximum landslide are given below.

The primary landslide parameters that are used in the tsunami models include the excavation depth and the slide width, which can be directly measured from sea floor mapping of the largest observed slide in the four geologic provinces. The other necessary parameter is the downslope landslide length, which is interpreted from the runout distance. The runout distance measured from sea floor mapping is a combination of fast plug flow (low viscosity, non-turbulent); creeping plug flow (high viscosity/viscoplastic; non-turbulent); and turbidity currents (turbulent boundary layer fluid). The latter two likely have little to no tsunami-generating potential. The landslide lengths indicated below are intended to represent the main tsunami-generating phase. The amplitude of the initial negative wave above the excavation region is linked to the maximum excavation depth. The amplitude of the initial positive wave above the deposition region is determined from a conservation of landslide volume. The excavation volume can be determined using GIS techniques (see below). Setting the deposition volume equal to the excavation volume determines the positive amplitude for a given landslide length. For a fixed volume, increasing the landslide length decreases the initial positive amplitude of the tsunami.

Landslide volume calculations are based on measuring the volume of material excavated from the landslide source area using a technique similar to that of ten Brink et al., (2006) and Chaytor et al., (2009). Briefly stated, the approach involves using multibeam bathymetry to outline the extent of the excavation area, interpolating a smooth surface through the polygons that define the edges of the slide to provide an estimate of the pre-slide slope surface, and subtracting this surface from the present seafloor surface.

The maximum observed landslide from multibeam surveys is taken as the maximum landslide for a given region. It may be possible that larger landslides could occur in a given region. However, this determination of the maximum landslide is consistent with the overall definition of the PMT as “the most severe of the natural phenomena that have been historically reported or determined from geological and physical data for the site and surrounding area.” In this case, the maximum landslide is taken from geologic observations spanning tens of thousands of years.

a. <u>*East Breaks Landslide*</u>

Geologic Setting: The river delta that formed at the shelf edge during the early Holocene.

Age: 10,000 to 25,000 years.

Maximum Single Event: Maximum and minimum parameters are taken from different interpretations of the digitized failure scar surrounding the excavation region (Chaytor et al., 2009).

Volume	**Area**	**Width**	**Length**	**Excavation Depth**	**Runout Distance**
Max: 21.95 km^3 Min: 20.80 km^3	519.52 km^2 420.98 km^2	~ 12 km	~ 50 km	~160 m	91 km*
*From the toe of the excavation area and 130 km from the headwall based on GLORIA data. Note that the multibeam bathymetry is not available for the entire runout area.					

b. <u>*Mississippi Canyon*</u>

Geologic Setting: River delta and fan system.

Age: 7,500 to 11,000 years.

<u>Maximum Single Event</u>

Volume	**Area**	**Excavation Depth**	**Runout Distance**

425.54 km^3	3687.26 km^2	~300 m	297 km*
*From the toe of the excavation area and 442 km from the headwall scarp.			

c. *Florida Escarpment*

Geologic Setting: Edge of a carbonate platform.

Age: Early Holocene or older. Because the Florida Escarpment carbonate failures are buried by Mississippi Fan deposits, the Florida Escarpment failures are older than the youngest fan deposits dated at about 11,500 years old.

Maximum Single Event

Volume	Area	Excavation Depth	Runout Distance
16.2 km^3	647.57 km^2	~150 m, but quite variable	Uncertain*
*The landslide deposit is at the base of the Florida Escarpment and is buried under younger Mississippi Fan deposits.			

d. *Campeche Escarpment*

Geologic Setting: Carbonate platform.

Age: No specific data are available

Maximum Event: *No specific data are available nor obtainable because the East Break is located within the territory of Mexico.* One of the persistent issues during the independent confirmatory analysis is acquiring sufficient geologic information about the Campeche Escarpment with which to estimate the maximum landslide parameters, as with the other Gulf of Mexico landslide provinces. Plans to conduct multibeam bathymetric surveys are pending. Presently, there is no published information showing the detailed bathymetry or distribution of landslides on or above the Campeche Escarpment.

Seismic Seiches

Rather than being impulsively generated by the displacement of the sea floor, seismic seiches occur from the resonance of seismic surface waves within enclosed or semi-enclosed bodies of water. The harmonic periods of the oscillation are dependent on the dimensions and geometry of the body of water. In 1964, seiches were set up along the Gulf Coast from seismic surface waves emanating from the M = 9.2 Gulf of Alaska earthquake, owing (in part) to the amplification of seismic waves from the thick sedimentary section along the Gulf Coast. Because the propagation path from Alaska to the Gulf Coast is almost completely continental, and because the magnitude of the 1964 earthquake is close to the maximum possible for that subduction zone, it is likely that the historical observations of the 1964 seiche wave heights are the maximum possible and less than the PMT amplitudes from landslide sources.

In summary, the discussion that follows is a list of the findings in the NRC staff's independent confirmatory analysis of the tsunami source characteristics:

- There is sufficient evidence to consider submarine landslides in the Gulf of Mexico a present day tsunami hazard for the purpose of defining the PMT at the STP site.

- Four geologic landslide provinces are defined in the Gulf of Mexico that are applicable for determining the PMT: northwest of the Gulf of Mexico, the Mississippi Canyon, the Florida Escarpment, and the Campeche Escarpment. The propagation paths that result in the least attenuation of potential tsunamis are the East Breaks and the Campeche provinces.

- Parameters for the maximum submarine landslide were determined for each of the provinces, except for the Campeche Escarpment (which is awaiting additional data).

- It is likely that seismic seiche waves resulting from the 1964 Gulf of Alaska earthquake are nearly the highest possible owing to a predominantly continental ray path for seismic surface waves from Alaska to the Gulf Coast.

2.4S.6.4.3 Tsunami Analysis

Information Submitted by Applicant

Based on the review of tsunami sources, the applicant indicates that modeling tsunami wave heights and periods at the site is not warranted and was not performed. However, the applicant conducted a tsunami analysis in response to RAI 02.04.06-1.

NRC Staff's Technical Evaluation

The most common computational models include MOST, COMCOT, and TSUNAMI2. All three models solve the same depth-integrated and 2D-horizontal (2DH) nonlinear shallow-water (NSW) equations with different finite-difference algorithms. There are a number of other tsunami models, including the finite element model ADCIRC.

Earthquake-generated tsunamis, with their very long wavelengths, are ideally matched with NSW equations for transoceanic propagation. Models such as MOST and COMCOT have been shown to be reasonably accurate throughout the evolution of a tsunami and are in widespread use today. However, when examining the tsunamis generated by submarine mass failures, the NSW equations can lead to significant errors (Lynett et al., 2003). The length scale of a submarine failure tends to be much less than that of an earthquake, and thus the wavelength of the created tsunami is shorter. To correctly simulate the shorter wave phenomenon, there needs to be equations with excellent shallow to intermediate water properties, such as the Boussinesq equations. Thus, for the work proposed here, the Boussinesq-based numerical model COULWAVE (Lynett and Liu, 2002) will be used. For technical details on wave propagation, breaking, runup, inundation, and overtopping of sloping structures see Geist et al., (2009) (including the references).

In response to RAI 02.04.06-1, the applicant models a tsunami from the East Breaks landslide using a NSW wave model (MOST) that is described in FSAR Version 3.0. In contrast, NRC staff used a higher-order Boussinesq hydrodynamics model (COULWAVE) in the staff's confirmatory analysis. This model is more specifically suited to landslide tsunamis.

2.4S.6.4.4 Tsunami Water Levels

Information Submitted by Applicant

To determine the maximum tsunami water levels, the applicant used a published 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 runup amplification factor, (2) 10 percent exceedance of an astronomical high tide, and (3) sea level rise from global climate change. The applicant's finding for the PMT maximum water level is 11.5 feet above MSL, which includes the effects of the high tide exceedance and sea level rise in the next century on the site.

NRC Staff's Technical Evaluation

An independent confirmatory analysis of tsunami water levels at the STP site focuses on distant earthquake tsunami sources and landslide sources local to the Gulf of Mexico.

a. Distant Earthquake Sources

For comparative purposes, NRC 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 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. These results confirm that tsunami amplitudes from distant Caribbean earthquakes are less than 1.0 m (3.28 ft) near the STP site. Tsunami amplitudes from earthquakes along the Azores-Gibraltar oceanic convergence boundary are also likely to be less than 1 m (3.28 ft) in the Gulf of Mexico. Therefore the staff finds the applicant's analysis acceptable.

b. Local Landslide Sources

A detailed tsunami analysis was performed for two local landslide scenarios: (1) the East Breaks landslide, and (2) a hypothetical landslide along the Campeche Escarpment. For each case, COULWAVE was used to compute the tsunami propagation, runup, and inundation (see Subsection 2.4S.6.4.4). For the development of the numerical grid and for additional details, see Geist et al., (2009). Therefore the staff finds the applicant's analysis acceptable.

Initial Numerical Simulations – Physical Limits

The purpose of these initial staff simulations are to provide an upper limit of the tsunami wave height that could be generated by the Gulf of Mexico landslide scenario. Source parameters for the simulation include landslide width, length, and excavation depth. Although the landslide volume is not a direct parameter that was used in the model, the volumes of excavation and deposition were conserved and used to determine the amplitude of the initial positive wave. Note that these limiting simulations used physical assumptions that are arguably unreasonable. The results of these simulations will be used to filter out tsunami sources that are incapable of adversely impacting the STP site under even the most conservative assumptions. Specifically, these staff assumptions are:

1. The time scale of the submarine landslide motion is very small (i.e., instantaneous) compared to the period of the generated tsunami.

2. Bottom roughness and the associated energy dissipation are negligible in locations that are initially wet (i.e., locations with a negative bottom elevation offshore).

With Assumption 1, the free-water surface response matches the change in the seafloor profile exactly. The landslide time-evolution parameter, which is associated with a high degree of uncertainty, is thus removed. Assumption 2 prevents the use of an overly high bottom-roughness coefficient, which could artificially reduce the tsunami energy reaching the shoreline. Such an assumption is too physically unrealistic to accept for the inland regions, where the roughness height may be the same order as the flow depth. For tsunami inundation, particularly for regions similar to the location of this project where the wave would need to inundate long reaches of densely vegetated land to reach the site, it is necessary to include a conservative measure of bottom roughness.

<u>East Breaks Landslide</u>

<u>1HD Results</u>: For the East Breaks landslide, both 1 and 2 horizontal-dimension (HD) simulations were performed. The 1HD simulations do not include radial spreading (representing the extreme case of an infinitely wide landslide) and refraction effects. Refraction can have either a constructive or destructive effect on the wave height, depending on the shallow water depth contours.

Three 1HD simulations were performed for cases of varying on-shore bottom friction: (a) bottom friction due to the small roughness characteristic of a very smooth and sandy ground (bottom-drag coefficient, $f = 0.001$); (b) bottom friction due to the small/moderate roughness characteristic of grass/turf ($f = 0.01$); and (c) bottom friction due to the large roughness characteristic of the trees and the dense, shrub-like vegetation that currently exists seaward of the STP reservoir ($f = 0.05$).

The Low Friction Case “a” shows a fast-moving bore front that easily overtops the STP main cooling reservoir, with maximum water surface elevations approaching about 98 ft (30 m). Despite the relatively low friction value used in Case “b,” the tsunami wave front slows significantly here. The wave does not overtop the main cooling reservoir, and maximum water elevations near the STP site are approximately 33 ft (10 m) (Figure 2.4S.6.4.5). Finally, for Case “c,” the large realistic friction retards the flow considerably, and the tsunami wave front does not reach the STP site but still manages to travel 10 km (6.22 mi) inland. A conclusion of this 1HD East Breaks study is that a tsunami approaching the site, with a bore height up to about 30 m (98 ft) at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for. Again, the 1HD case does not include the lateral dissipation (radial spreading) of the wave from the source.

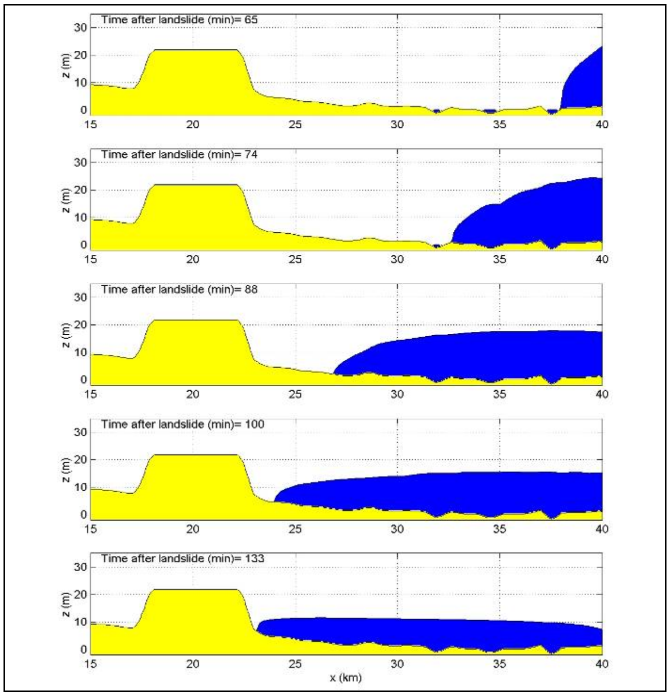

Figure 2.4S.6.4.5-1. The Onshore Evolution of the 1HD Tsunami from the East Breaks Scenario for the Mid-Friction Case (Case "b"). A Cross-Sectional Profile of the Main Cooling Reservoir is Shown on the Left Side

2HD Results: The 2HD simulation provides information about the importance of radial spreading and refraction, which can be used to qualitatively correct the 1HD results. With no refractive amplification and significant radial spreading, the 2HD tsunami height is less than the 1HD near the shoreline, with the 2HD simulation yielding bore height predictions on the order of about 10 m (33 ft) at the shoreline, or 1/3 of the 1HD prediction. Considering this 2HD spreading reduction with the 1HD inundation results and the conservative "hot-start" approach that the simulation employed, it can be stated with high certainty that the tsunami from the East Breaks landslide will not impact the STP site.

Uncertainty in the primary landslide source parameters for the tsunami (excavation depth and slide length) is, to a great extent, diminished owing to the depth-limiting effects on the amplitude during propagation across the south Texas continental shelf. Depth-limiting effects mean that for a given beach profile and incident wave period, there is some ratio of wave height to shelf water depth that remains more or less constant, as the wave propagates across the broad continental shelf.

Campeche Landslide

Presently, there is no available information showing the detailed bathymetry or the distribution of landslides on or above the Campeche Escarpment. As a provisional source for the Campeche Escarpment, NRC staff used initial conditions applicable to the maximum observed landslide along the Florida Escarpment (a similar geologic environment). The Campeche Escarpment is includes an initial drawdown of 150 m (492 ft), with a horizontal length scale of 20 km (12.43 mi). The very steep slope of the Campeche Escarpment results in the maximum depression occurring over a depth of 500 m (1,640 ft), whereas the maximum positive wave of the initial condition occurs over a depth of 1,000 m (3,281 ft). Because the propagation distance for Campeche is much larger than that of East Breaks (about 700 km [435 mi] longer), the two-dimensional spreading effect will likely be very significant and will result in a greater attenuation than for the East Breaks scenario. For the 2HD setup, slide widths of 20 km and 60 km (12.43 mi and 31.08 mi) were tested. The former is the expected maximum for the Florida Escarpment; the latter is similar to the maximum width in the Storegga landslide complex and the width for the "Monster" scenario landslide that the applicant used for the south Texas continental shelf. In both cases, the wave heights decrease very quickly near the source, but they reach a nearly steady (slowly attenuating) condition when they reach the continental shelf off the Gulf Coast. SER Figure 2.4S.6.4.5-2 shows a cross section, with the waves taken from the 2HD slide for the Campeche 60-km (37.29-mi) slide at the time of maximum inundation (Mid-Friction Case "b"). The general conclusion reached after comparing the East Breaks scenario with the Campeche scenario is that given the level of uncertainty in the source parameters, the approaching wave heights for the hypothetical Campeche scenario are comparable to those of the East Breaks scenario.

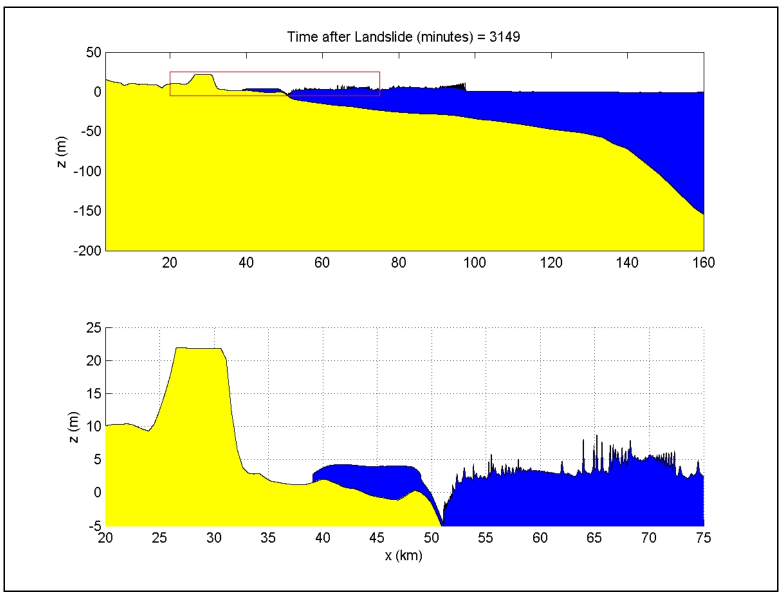

Figure 2.4S.6.4.5-2. Wave Profile at the Time of Maximum Inundation for the Campeche 2HD 60-km Slide Width Source Scenario and for the Mid-Friction Case (Case "b") (Top) View Across the Continental Shelf (Bottom) View Near the STP Site

An independent analysis of the 10 percent exceedance high tide was conducted for 16 years of NOAA National Ocean Service Center for Operational Oceanographic Product Services (NOS-CO-OPS) data at the Freeport tide gauge station (years 1992 through 2007), (NOAA, 2008). The 10 percent exceedance high tide was determined to be 0.45 m (1.48 ft) relative to the MSL for these years. This finding is consistent with the applicant's estimate of 0.46 m (1.51 ft) relative to the MSL and is indicated in the FSAR, but the number is inconsistent with the estimated 1.08 m (3.54 ft) indicated in the response to RAI 02.04.06-1 (ML083460084). The long-term sea level rise at the Freeport station is 4.35 ± 1.12 mm/year (yr) (0.17 ± 0.04 in./yr), according to the NOAA NOS-CO-OPS data and also indicated in the applicant's RAI response. The estimate in the applicant's FSAR is 5.87 ± 0.74 mm/yr (0.23 ± 0.03 in./yr). Therefore, the PMT water level for the conservative 2HD tsunami during the next century is 4 m

(13.12 ft) maximum tsunami runup + 0.45 m (1.47 ft) (10 percent exceedance high tide) plus 0.59 m (1.94 ft) (century sea level rise) or approximately a sum of 5.04 m (16.53 ft).

Results of the analysis indicate that the PMT source is a submarine landslide, either along the continental slope directly across from the site (i.e., East Breaks scenario) or along the Campeche Escarpment. There is a high degree of uncertainty in the source parameters for the latter scenario. Hot-start initial conditions were used to represent conservative values related to tsunami generation efficiency. In addition, several bottom-friction parameters for overland flow were tested representing realistic and conservative estimates. Realistic wave propagation in the 2HD simulation, yielded the PMT runup of approximately 5 m (16.44 ft) (relative to the MSL) for conservative hot-start initial conditions and conservative values of bottom friction for overland flow, considering the effects of a 10 percent exceedance high tide and sea level rise during the next century.

2.4S.6.4.5 Effects on Safety-Related Facilities

Information Submitted by Applicant

Because the maximum tsunami water level associated with the PMT is below grade elevations at the site, the applicant concludes that there will be no onsite tsunami waves affecting safety-related facilities.

NRC Staff's Technical Evaluation

NRC staff concurred with the applicant that because the maximum tsunami water level associated with the PMT is below grade elevations at the site, there will be no onsite tsunami waves affecting safety-related facilities.

2.4S.6.5 Post Combined License Activities

There are no post COL activities related to this subsection.

2.4S.6.6 Conclusion

NRC staff reviewed the applicant's submittals in FSAR Section 2.4S.6 and in response to the RAIs. As set forth above, the applicant presents and substantiates sufficient information pertaining to estimates of the effects from probable maximum tsunami hazards at the proposed STP site, and no outstanding information is required to be addressed in the COL FSAR for this section. Furthermore, the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the probable maximum tsunami hazards, with a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data were accumulated.

NRC staff accepted the methodologies used to determine the severity of the tsunami phenomena reflected in this analysis, as documented in this SER section. In the context of the above discussion, the applicant's analysis is acceptable for use in establishing the design bases for SSCs important to safety, as may be proposed in a COL or CP application. Accordingly, the staff concluded that the use of these methodologies results in an analysis containing a sufficient margin for the limited accuracy, quantity, and period of time in which the data were accumulated. Moreover, the PMT surge level estimates by both the applicant and the staff are far below the bounding main cooling reservoir breach water level of 12.2 m (40 ft) MSL or the plant grade of 10.36 (34 ft) MSL, thus the staff concluded that the postulated PMT would not affect the proposed STP site.

Therefore, the staff found that the identification and consideration of the PMT hazards set forth above are acceptable and meet the requirements of 10 CFR 52.79(a)(1)(iii), 10 CFR 100.20(c), and 10 CFR 100.23(d)(3). The information addressing COL License Information Item 2.14 is adequate and acceptable.

2.4S.6.7 References

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2.4S.7 Ice Effects

2.4S.7.1 Introduction

This section of the FSAR addresses the ice effects to ensure that safety-related facilities and water supply are not affected by ice-induced hazards.

This SER section presents an evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: ice conditions and historical ice formation, ice jam events, the effect of ice on cooling-water system, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.79(a).

2.4S.7.2 Summary of Application

In Section 2.4S.7, the applicant addresses the information related to the site ice effects. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.16 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.16 Ice Effects

This section addresses the COL-specific information identified in DCD Tier 2, Revision 4, Section 2.3. COL License Information Item 2.16 requires the COL applicants to demonstrate that safety-related facilities and the water supply are not affected by ice flooding or blockage. This information is provided below.

2.4S.7.3 Regulatory Basis

The relevant requirements of the Commission regulations for the identification and evaluation of ice effects, and the associated acceptance criteria, are in Section 2.4.7 of NUREG-0800.

The applicable regulatory requirements for identifying ice effects are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3) sets forth the criteria to determine the siting factors for plant design bases with respect to flood level and wave action at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices

2.4S.7.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.7 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to the site ice effects. The staff’s technical review of this section includes an independent review of the applicant’s information in the FSAR and in the responses to the RAIs.

This section of the SER provides the staff’s evaluation of the technical information presented in FSAR Section 2.4S.7.

COL License Information Item

- COL License Information Item 2.16 Ice Effects

The staff reviewed the applicant’s information in FSAR Section 2.4S.7. The staff independently assessed the potential for formation of ice at the STP site using available data. The staff’s evaluation is described below.

2.4S.7.4.1 Ice Conditions and Historical Ice Formation

Information Submitted by Applicant

The applicant uses long-term daily air temperature data from onsite measurements (1990-2006) and from the Bay City station (1942–2006) to assess the potential for ice formation near the STP site. The maximum cumulative degree-day is a measure of severity of site-specific winter weather conditions conducive to ice formation. The applicant states that, in the observed daily air temperature records at the site, there was only one instance in 1983 when the average daily air temperature was below freezing for five consecutive days. The applicant also states that at the Bay City station there are two instances (1973 and 1989) when the average daily air temperature was below freezing for four consecutive days, and three instances (1948, 1951, 1963, 1985) when the average daily air temperature was below freezing for three consecutive days. Based on these data, the applicant concludes that conditions conducive to freezing rarely occur near the site, and these rare occurrences are of a very short duration.

LCRA recorded water temperature data from 1982 through 2006 at three stations: Bay City (Site 12284), Wharton (Site 12286), and Columbus (Site 12290), which are located approximately 22.5, 59.5, and 114.3 km (14, 37, and 71 mi) from the STP site, respectively. The applicant uses the LCRA recoded data to determine the minimum water temperatures in the Colorado River near the STP site. The minimum recorded water temperature in this data set is 5.1 ºC (41.2 ºF) on February 6, 1985. At the intake within the main cooling reservoir, water temperatures ranged from 10.6 to 33.4 ºC (51.1 to 92.1 ºF) based on measurements between 1997 and 2005. Based on these data, the applicant concludes that there is no risk of ice formation near the STP site.

NRC Staff's Technical Evaluation

The staff independently analyzed air temperature data downloaded from the National Climatic Data Center (NCDC) web site (NCDC 2008a, 2008b, 2008c) for three NCDC cooperative stations: Bay City Water Works (Coop ID 410569), Matagorda 2 (Coop ID 415659), and Palacios Municipal Airport (Coop ID 416750). The data at the Bay City station span the periods from October 1909 through July 1917 and from July 1942 through December 2008. The data at the Matagorda station span the period from July 1910 through December 2008. The data at the Palacios station span the period from February 1943 through February 2009. The staff analyzed these data to determine several parameters related to low air temperatures at these stations. These parameters are summarized in Table 2.4S.7-1 below.

Table 2.4S.7-1. Statistics of Low Air Temperatures Near the STP Site

Statistics	Bay City	Matagorda	Palacios
Lowest daily mean air temperature	-8.6 ºC (16.5 ºF) on 12/23/1989	-7.5 ºC (18.5 ºF) on 12/23/1989	-8.1 ºC (17.5 ºF) on 12/23/1989
Number of days with daily mean air temperature below freezing	83 of 24,530	59 of 28,820	63 of 23,934
Longest period with daily mean air temperature at or below 32 ºF (occurrences)	5 (twice)	5 (once)	5 (once)
Longest period with daily mean air temperature at or below 18 ºF (occurrences)	1 (twice)	0 (none)	1 (once)
ºC = degrees centigrade; ºF = degrees Fahrenheit.			

Based on the above analysis, the staff concluded that the mean air temperature near the STP site occasionally falls below freezing. However, these spells do not last more than 5 consecutive days. Frazil ice forms in turbulent, supercooled water that is not covered by an ice layer but is directly in contact with the atmosphere when the air temperature is below -7.8 ºC (18 ºF) (USACE, 2002). The daily mean air temperature at or below -7.8 ºC (18 ºF) was not sustained for more than a day. The staff concluded that ice formation near the STP site is an unlikely event. The staff also concluded that because of the lack of sustained air temperatures below -7.8 ºC (18 ºF), frazil ice formation is unlikely near the STP site.

2.4S.7.4.2 Ice Jam Events

Information Submitted by Applicant

The applicant states that there are no records of ice jams on the Lower Colorado River in the USACE Ice Jam Database. Review of the water temperature data from 1982 through 2006 shows that water temperatures in the Lower Colorado River never approach freezing. Therefore, the applicant notes that the formation of frazil and anchor ice at the RMPF is highly unlikely. The applicant also states that existence of large dams upstream of the site reduces the possibility that any surface ice or ice flows will move downstream to the STP site.

NRC Staff's Technical Evaluation

The staff searched the USACE Ice Jam Database to locate ice jam and ice dam events on the Colorado River (USACE, 2008). There is only one ice jam event listed in the database, and that jam is on the Brazos River at Rainbow, Texas. The weather bureau reported that in 1940, the Brazos River was obstructed by rough ice on January 22 through 23, January 25 through 27, and January 25 through 28. However, there are no records of any ice jam or ice dam formation on the Colorado River in the database.

Based on the Ice Jam Database search, the staff determined that the formation of ice jam and ice dam in the Colorado River near the vicinity of the STP site has never been observed. Therefore, the staff concluded that the formation of ice jam and ice dam near the STP site is an unlikely event.

2.4S.7.4.3 Effect of Ice on Cooling-Water Systems

Information Submitted by Applicant

The applicant states that the UHS for each of STP Units 3 and 4 consists of mechanical draft cooling towers and water-storage basin. The storage basin contains a sufficient capacity to supply water for 30 days following a design-basis accident without the storage basin receiving any makeup water.

The applicant states that the UHS and RSW systems remove heat from the closed-loop reactor building cooling-water system during normal, hot standby, normal shutdown, startup, loss of preferred power, and emergency shutdown operating modes. The UHS is also designed to bypass the cooling towers during cold weather operation. Ice formation in the UHS basin is not expected because of the temperate climatic condition near the site and because of the fact that it is always in service during the above operating modes.

NRC Staff's Technical Evaluation

The staff concurred with the applicant that the storage basin of the UHS is the only safety-related system that could be affected by ice formation. The applicant states that the UHS system will be designed to bypass the cooling tower and use the UHS water-storage basin directly during the cold weather operation. Continuous use of the UHS also reduces the possibility of ice formation within the UHS water-storage basin owing to the emitted heat in the cooling water. Therefore, the staff concluded that ice effects on the UHS are not significant.

2.4S.7.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.7.6 Conclusion

The staff performed an independent analysis to determine that ice and frazil formation near the STP site is unlikely. The staff also determined that no historical ice jam or ice dam formation in the Colorado River has been observed upstream or downstream of the site. The staff determined that brief freezing spells near the STP site would not affect the safety-related UHS operation.

As set forth above, the applicant presents and substantiates information relative to the ice effects important to the design and siting of the proposed plant. The staff found that the applicant has considered the appropriate site phenomena for establishing the design bases for SSCs important to safety. The staff accepted the methodologies used to determine the potential for ice formation and blockage reflected in these site characteristics, as documented in SERs for previous licensing actions. Accordingly, the staff concluded that the use of these methodologies results in site characteristics with a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

Based on the above review, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL License Information Item 2.16 is adequate and acceptable.

2.4S.7.7 References

National Climatic Data Center, "Bay City Wtr Wks, Bay City, TX," 2008a; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024300; accessed April 17, 2008.

National Climatic Data Center, "Matagorda 2, Matagorda, TX," 2008b; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024268; accessed April 17, 2008.

National Climatic Data Center, "Palacios Municipal Airport, Palacios, TX," 2008c; http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20024269; accessed April 17, 2008.

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.

U.S. Army Corps of Engineers, "Engineering and Design – Ice Engineering," Engineer Manual 1110-2-1612, Department of the Army, Washington, D.C., 2002.

U.S. Army Corps of Engineers, "Ice Jam Database," Ice Jam Information Clearinghouse, Cold Regions Research and Engineering Laboratory, 2008; http://www.crrel.usace.army.mil/icejams/; accessed April 17, 2008.

2.4S.8 Cooling-Water Canals and Reservoirs

2.4S.8.1 Introduction

This section of the FSAR addresses the cooling-water canals and reservoirs used to transport and impound water supplied to the safety-related SSCs. This SER section presents an evaluation of the design basis for the capacity and operating plan for safety-related cooling-water canals and reservoirs, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.8.2 Summary of Application

In FSAR Section 2.4S.8, the applicant provides site-specific information related to the cooling water canals and reservoirs. In addition, in this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.17 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed the information related to cooling-water canals and reservoirs as follows:

COL License Information Item

- COL License Information Item 2.17 Cooling-Water Channels and Reservoirs

COL License Information Item 2.17 requires the COL applicants to provide the basis for the hydraulic design of channels and reservoirs used to transport and impound plant cooling and to protect safety-related structures.

2.4S.8.3 Regulatory Basis

The relevant requirements of the Commission regulations for the cooling water canals and reservoirs, and the associated acceptance criteria, are in Section 2.4.8 of NUREG–0800.

The applicable regulatory requirements for cooling-water canals and reservoirs are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave actions at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”
- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices
- RG 1.102, “Flood Protection for Nuclear Power Plants”

2.4S.8.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.8 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to the cooling-water canals and reservoirs. The staff’s technical review of this section includes an independent review of the applicant’s information in the FSAR and in the responses to the RAIs.

This section describes the staff’s evaluation of the technical information presented by the applicant in FSAR Section 2.4S.8.

COL License Information Item

- COL License Information Item 2.17 Cooling-Water Channels and Reservoirs

The staff reviewed the applicant’s supplemental information relating to the cooling-water canals and reservoirs at the STP site and vicinity. The staff’s review of the application is summarized below.

2.4S.8.4.1 Cooling-Water Canals

Information Submitted by Applicant

The circulating-water intake structure for STP Units 3 and 4 will be located on the north dike within the main cooling reservoir. The circulating-water discharge structure for STP Units 3 and 4 will be located on the west side of the circulating-water discharge structure for STP Units 1 and 2 on the north embankment of the main cooling reservoir (see FSAR Figure 2.4S.8-1). The main cooling reservoir has two non-safety-related channels, the discharge and intake channels, which were originally designed for four reactor units. Each of the channels has a bottom width of 304.8 m (1,000 ft) and a side slope of 3:1 (horizontal versus vertical or H:V). The bottom elevation of the intake channel varies from 6.2 to 6.7 m (20.5 to 22.0 ft) MSL, and the bottom elevation of the discharge channel is 6.7 m (22.0 ft) MSL. The intake channel will be locally modified to accommodate the approach channel for the new STP Units 3 and 4 intake structure. No modification of the discharge channel will be necessary. The applicant states that, because the intake and discharge channels are submerged, they are not subject to wind-wave activity.

The spillway channel of the main cooling reservoir delivers any discharge from the reservoir over the spillway to the Colorado River. The channel has a length of approximately 1,591 m (5,220 ft), a width of 36.6 m (120 ft), an average depth of 3.7 m (12 ft), a longitudinal slope of approximately 0.2 percent, and a side slope of 5:1 H:V. No changes to this channel will result from the addition of STP Units 3 and 4. The applicant states that the operation of the main cooling reservoir spillway channel is nonsafety-related.

The existing RMPF that provides makeup water from the Colorado River to the main cooling reservoir will be shared among all four STP units. The RMPF is not a safety-related facility. The RMPF will be upgraded to include additional pumps, screens, and rakes to accommodate the additional makeup water demand for STP Units 3 and 4. The addition of STP Units 3 and 4 will not change the original makeup intake design flow rate of about 34 m^3/s (1,200 cfs).

NRC Staff's Technical Evaluation

The staff determined that the applicant has appropriately identified and described all cooling-water channels. Because there are no safety-related canals proposed for STP Units 3 and 4, the staff omitted the evaluation of these canals.

2.4S.8.4.2 Reservoirs

Information Submitted by Applicant

There are two reservoirs on the STP site: the 28.3 km^2 (7,000-ac) main cooling reservoir, which will be shared among all four units and is part of their closed-loop cooling system; and the 186,152 m^2 (46-ac) ECP, which serves as the UHS for STP Units 1 and 2. The ECP will not be affected by the construction of STP Units 3 and 4 and has no function in their operation.

The main cooling reservoir will be part of the closed-loop CWS to dissipate heat produced from all four units during their normal operations. The Colorado River, via the RMPF, will provide makeup water to the main cooling reservoir to replace water losses due to evaporation, seepage, and blowdown. The main cooling reservoir is enclosed by a compacted clay-filled embankment with an exterior slope of 3:1 H:V and an interior slope of 2.5:1 H:V, with a 76.2 cm (30-in.) thick soil cement lining to prevent erosion. The top of the embankment varies in elevation from 20.1 to 20.5 m (65.8 to 67.1 ft) MSL. An interior dike, constructed of compacted clay lies within the main cooling reservoir to prevent the short-circuiting of discharged warm water to the intake. The reservoir side of the main cooling reservoir embankment and both sides of the interior dike are lined with 76.2-cm (30-in.) -thick soil-cement to protect against erosion from the wave action. The outside of the peripheral embankment is sodded for erosion protection.

Except for June 6, 1985, when the main cooling reservoir water surface elevation was at 8.4 m (27.7 ft) MSL during its initial filling, the minimum water surface elevation in the main cooling reservoir was 10.2 m (33.4 ft) MSL on November 11, 1987; and the maximum water surface elevation in the main cooling reservoir was 14.5 m (47.6 ft) MSL on July 3, 2003. The normal maximum operating water level for STP Units 1 and 2 is 14.3 m (47.0) ft MSL, which is less than the design normal maximum operating level of 14.9 m (49.0 ft) MSL for the reservoir. The applicant states that, when all four units are in operation, the normal maximum operating water surface elevation will be maintained at 14.9 m (49.0 ft) MSL.

<u>New CWS Intake and Discharge Structures</u>

The new CWS intake structure for STP Units 3 and 4 will be approximately 40 m (130 ft) long and 122 m (400 ft) wide. It will be located on the east slope of the interior dike approximately 107 m (350 ft) south of the existing STP Units 1 and 2 CWS intake structure. The new discharge structure for STP Units 3 and 4 will be located on the north embankment of the main cooling reservoir, approximately 305 m (1,000 ft) west of the existing discharge structure. The new structure will be approximately 18.3 m (60 ft) long and 61 m (200 ft) wide. The applicant states that neither structure is safety-related.

Spillway

The main cooling reservoir spillway is located at its southeast corner and is used to release any water exceeding the normal maximum operating storage. The spillway is a gated concrete ogee with the crest at 12.2 m (40.0 ft) MSL. Four 1.8-m (6-ft) wide and 2.9-m (9.5-ft) tall gates are located on top of the ogee crest. The spillway is not a safety-related structure.

To check the safety of the embankment from overtopping, the applicant estimates the maximum water surface elevation within the main cooling reservoir during a local PMP event (STPEGS, 2006). The applicant routes the 72-hour storm total precipitation input of 141.5 cm (55.7 in.) through the main cooling reservoir accounting for area and storage curves of the reservoir, operating procedures of the main cooling reservoir spillway, and rating curve of the spillway. The applicant sets the initial water surface elevation in the main cooling reservoir to the normal operating water level of 14.9 m (49 ft) MSL. The applicant estimates the maximum water surface elevation in the main cooling reservoir to be 16 m (52.6 ft) MSL, which is significantly lower than the lowest top elevation of the main cooling reservoir embankment of 20 m (65.8 ft).

Embankment Freeboard

The applicant estimates the embankment freeboard using the PMH sustained wind speeds adjusted to overland wind speeds, as described by the NWS (1979) at eight locations within the main cooling reservoir. The applicant estimates wave height, runup, and wind setup elevations using the methods described by the USACE (2008). The applicant states that the waves are not limited by water depth. Under local PMP-induced flooding in the main cooling reservoir, the applicant estimates the stillwater elevation to be 16 m (52.6 ft) MSL. As recommended in ANSI/ANS-2.8–1992 (ANS, 1992), the applicant also estimates wind waves induced by a 2-year wind wave and adds them to the stillwater elevation to obtain a final water surface elevation of 17.79 m (58.38) ft MSL, which is significantly below the lowest top elevation of the main cooling reservoir embankment. Therefore, the applicant concludes that there is sufficient freeboard at the main cooling reservoir.

Seiche in Main Cooling Reservoir

The applicant assumes the PMH passing over the reservoir as the forcing mechanism for a seiche in the main cooling reservoir. The applicant estimates significant wave height induced by the PMH winds to be approximately 4 m (13 ft), with a spectral wave period of 4.7 seconds. The applicant estimates the natural frequency of the main cooling reservoir to be approximately 22 minutes. Because the spectral wave period of the PMH-generated wind waves is significantly smaller than the natural frequency of the main cooling reservoir, the applicant concludes that the energy of the PMH-generated waves will dissipate due to frictional losses and the raised water surface will decrease after each oscillation.

NRC Staff's Technical Evaluation

The staff determined that the applicant has appropriately identified and described the main cooling reservoir and its facilities, which are not safety-related structures. The only safety-related water reservoirs proposed for STP Units 3 and 4 are the two engineered, partially buried UHS water-storage tanks (basins) (FSAR Figures 2.5S.4-49A through 2.5S.4-49D). The two UHS water-storage tanks, one for each proposed unit, will be located south of the respective units. Section 9.2.5 of the FSAR evaluates the capacity of these UHS water-storage tanks. The NRC staff determined that the these UHS water-storage tanks will be sufficient to meet 30 days of the UHS cooling requirements under design-basis accident conditions, without needing a makeup or blowdown. Therefore, the staff found the applicant's description of the reservoirs acceptable.

Embankment Freeboard

During the review of in the main cooling reservoir embankment freeboard, the staff issued RAI 02.04.08-1 requesting the applicant to provide details of estimates of wind setup, wave height, and runup elevations at eight locations along the main cooling reservoir embankment. In a letter dated August 27, 2008 (ML091910403), the applicant's response to RAI 02.04.08-1 states that there will be no physical changes to the main cooling reservoir as a result of the construction and operation of Units 3 and 4 that will affect the characteristics of wind-wave setup and runup. The applicant therefore notes that the original main cooling reservoir freeboard analysis carried out for the design of the main cooling reservoir embankment during the licensing of STP Units 1 and 2 is still valid. In addition, the applicant provides a re-analysis of the wave setup and runup estimates using two conservative scenarios as described below.

The first scenario is the combined event of a 72-hour local PMP over the main cooling reservoir coincident with the 2-year wind wave. By routing the excess water in the reservoir through the spillway, the applicant estimates the maximum reservoir level from the 72-hour PMP of 16 m (52.6 ft) MSL. Using the estimated stillwater level with 2-year wind and an average reservoir bottom elevation of 7 m (23 ft) MSL, the applicant estimates the maximum water level of 17.8 m (58.4 ft) MSL near the spillway and 17.77 m (58.3 ft) MSL at the northern embankment, respectively.

The staff reviewed the applicant's response and determined that the combined event and the method used to estimate the maximum water surface elevation within the main cooling reservoir for the first scenario are appropriate. The STP Units 1 and 2 FSAR, Revision 13 (STPEGS, 2006), states that the main cooling reservoir embankment elevation near the spillway and at the north embankment is 20.2 m (66.2 ft) MSL. Therefore, staff determined that the combined event of a 72-hour local PMP event and a 2-year wind wave will not overtop the main cooling reservoir embankment.

The second scenario consists of wind waves induced by PMH winds, with the starting water surface elevation in the main cooling reservoir at the normal operating level of 14.9 m (49 ft) MSL. The applicant states that this analysis was performed for the main cooling reservoir embankment freeboard design during the licensing of STP Units 1 and 2 (STPEGS, 2006). STPEGS obtained the PMH speed of from NWS Technical Report 23 (NWS, 1979) and adjusted the speed for the movement over land and subsequent open water in the main cooling reservoir. The resulting PMH speed was 66.2 m/s (148 mph).

Subsequently, STPEGS estimated the wind setup from the PMH using the approach described by Saville et al., (1962) and the corresponding wave runup using the approach described by USACE (1977). STPEGS estimated the maximum water surface elevation along the main cooling reservoir embankment to be 19.9 m (65.2 ft) MSL and noted that it occurs on the south embankment, where the embankment elevation is 20.4 m (66.9 ft) MSL. STPEGS noted that a water surface elevation of 19.3 m (63.4 ft) MSL along the north embankment, where the embankment elevation is 20.2 m (66.2 ft) MSL. Based on this information, the applicant states that the minimum available freeboard along the main cooling reservoir embankment for this scenario is 0.52 m (1.7 ft).

The applicant modified the FSAR text to reflect the revised analyses in FSAR Subsection 2.4S.8.2.3. The applicant states that FSAR Figures 2.4S.8-2 through 2.4S.8-5 will be deleted. In FSAR Revision 7, these figures are deleted. This change was tracked as **Confirmatory Item 02.04.08-1**, which is now closed.

The staff independently estimated the PMH from NWS Technical Report 23 (NWS 1979), as described in Section 2.4S.5 of this SER. The staff found that the maximum PMH wind speed computed with the SLOSH model near the location of the STP Units 3 and 4 power block is approximately 83.1 m/s (186 mph). The staff independently estimated the wind-wave setup and runup at three locations: the spillway, the south embankment, and the north embankment of the main cooling reservoir. The average depth of the main cooling reservoir is estimated as the difference between normal main cooling reservoir water surface elevation 14.9 m (49 ft) MSL and the average main cooling reservoir bottom elevation 7 m (23 ft) MSL. The staff used a PMH wind speed of 83.1 m/s (186 mph), an initial water surface elevation in the main cooling reservoir of 14.9 m (49 ft) MSL, an average water depth of 7.9 m (26 ft), and 2.5H:1V for the inner slope of the main cooling reservoir embankment. The staff estimated the wind-wave parameters using the USACE (2008) methods. The staff determined that the wind waves within the main cooling reservoir are fetch limited, and the PMH winds are also limited by water depth. USACE (2008) recommends limiting wave heights to 0.6 times the depth of the water body. Therefore, the staff estimated the PMH wind-wave height in the main cooling reservoir to be approximately 4.8 m (15.6 ft) (i.e., 0.6 × 7.9 m [26 ft]). The corresponding estimated wind setups and wave runups using USACE (2008) at the three locations are in Table 2.4S.8-1, below.

Table 2.4S.8-1. NRC Staff-Estimated PMH Wind Setup and Wave Runup at Three Locations Within the Main Cooling Reservoir

Location	Fetch (km) / (mi)	Depth-Limited Wave Height (m) / (ft)	Spectral Wave Period (second)	Wind Setup (m) / (ft)	Wave Runup (m) / (ft)	MSL Water surface Elevation (m) / (ft)
Spillway	5.5 / 3.4	4.8 / 15.6	4.35	0.98 / 3.2	3.41 / 11.2	19.3 / 63.4
North Embankment	5.8 / 3.6	4.8 / 15.6	4.42	1.04 / 3.4	3.47 / 11.4	19.4 / 63.8
South Embankment	5.3 / 3.3	4.8 / 15.6	4.29	0.94 / 3.1	3.35 / 11.0	19.2 / 63.1
MSL = mean sea level; km=kilometer; mi=mile; m=meter; ft=foot;						

The staff estimates of water surface elevations within the main cooling reservoir at the three locations are 19.3, 19.4, and 19.2 m (63.4, 63.8, and 63.1 ft) MSL, respectively. The corresponding top elevations of the main cooling reservoir embankment at these locations are

20.2, 20.2, and 20.4 m (66.2, 66.2, and 66.9 ft) MSL, respectively. Therefore, the staff concluded that the PMH wind waves within the main cooling reservoir would not overtop the main cooling reservoir embankment.

The STPEGS Units 1 and 2 UFSAR Subsection 2.4.8.2.3, “Embankment Freeboard,” lists the maximum water surface elevation along the south embankment as 65.2 ft MSL, under the effects of PMH winds acting on a normal main cooling reservoir stillwater surface elevation of 14.9 m (49 ft) MSL. STP Units 3 and 4 FSAR Subsection 2.4S.8.2.3, “Embankment Freeboard,” states that the maximum water level due to wave runup under PMH winds is an estimated 17.79 m (58.38 ft) MSL. The staff issued RAI 02.04.08-2 requesting the applicant to explain the difference between these two estimates.

In a letter dated August 27, 2008 (ML091910403), the applicant’s response to RAI 02.04.08-2 states that the maximum water surface elevation of 19.9 m (65.2 ft) MSL along the south embankment is estimated in the STPEGS Units 1 and 2 FSAR Subsection 2.4.8.2.3 and results from a combination of PMH wind waves on an initial main cooling reservoir stillwater elevation of 14.9 m (49 ft) MSL. The applicant also states that the maximum water surface elevation of 17.79 m (58.38 ft) MSL is estimated at the spillway location based on the combination of a 72-hour local PMP event over the main cooling reservoir (with the initial main cooling reservoir stillwater elevation at 14.9 m [49 ft] MSL) and 2-year winds.

Based on the review of the RAI response 02.04.08-2 and the result of an independent confirmatory analysis, the staff found that the applicant’s estimation of the wave setup and runup are adequate. Therefore, the staff concluded that there is sufficient freeboard at the main cooling reservoir and consider RAIs 02.04.08-1 and 02.04.08-2 closed.

<u>*Seiche in Main Cooling Reservoir*</u>

The staff estimated the spectral wave period of the PMH-induced wind waves within the main cooling reservoir to be approximately 4.4 seconds. The natural period of free oscillation in a rectangular basin of constant depth can be estimated as

$$T = \frac{2L}{\sqrt{gh}}$$

where

T = the period of seiche motion in seconds,

g = the acceleration resulting from gravity (9.8 m/s^2 [32.2 ft/s^2]),

L = the length of the idealized rectangular basin in m or ft, and

h = the depth of the idealized rectangular basin in m or ft (Wilson, 1972).

The staff used the fetch length to approximate L and the main cooling reservoir average depth to approximate h. The staff estimated the period to vary from 19.9 to 21.7 minutes at the three locations within the main cooling reservoir that were also used to estimate wind-wave setup and runup. Based on the large difference between the natural period of the main cooling reservoir and the spectral wave period of the PMH-induced wind waves, the staff concluded that resonance would not occur within the main cooling reservoir. Therefore, the staff concluded that a wind-induced seiche would not be set up for an extended duration.

Seismic forcing can also generate a seiche within a lake if (1) the period of the seismic wave matches the natural period of free oscillation of the lake, and (2) the seismic waves that have periods not matching the natural period of free oscillation of the lake but provide many cycles of motion over the duration the waves pass the site (Barberopoulou et al., 2006; Barberopoulou, 2008). For example, the magnitude 7.9 Denali, Alaska, earthquake of 2002 produced long waves of approximately 100-second periods that produced resonating seiches in lakes near Seattle, Washington (Barberopoulou, 2008).

Long or transverse seismic waves that produce horizontal movement can induce seiches within the main cooling reservoir. For example, seiches were set up along the Gulf Coast from seismic surface waves emanating from the magnitude 9.2 Gulf of Alaska earthquake in 1964, owing in part to the amplification of seismic waves from the thick sedimentary section along the Gulf Coast. It is likely that seismic seiche waves resulting from the 1964 Gulf of Alaska earthquake are nearly the highest possible (refer to this SER Subsection 2.4S.6.4.3), with no significant seismic sources nearby. Therefore, the staff concluded that further review of a seismically induced seiche in the main cooling reservoir is not warranted.

2.4S.8.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.8.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to the design basis for canals and reservoirs used to transport and impound water supplied to the SSCs. In particular, the staff performed an independent confirmatory analysis to determine the potential overtopping of the main cooling reservoir caused by hurricane surge and seiche effects. Based on this analysis, the staff concluded that the main cooling reservoir embankment would not be overtopped under PMH or seiche conditions.

The staff reviewed the information provided and, for the reasons given above, concluded that the applicant has provided sufficient details about the site description to allow a staff evaluation, as documented in Section 2.4S.8 of this SER. Accordingly, the staff concluded that the use of these methodologies results in site characteristics that have a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated.

Based on the above information and review, the staff concluded that the identified site characteristics meet the requirements of 10 CFR 52.79 and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL License Information Item 2.17 is adequate and acceptable.

2.4S.8.7 References

Barberopoulou, A., “A Seiche Hazard Study for Lake Union, Seattle, Washington,” *Bulletin of the Seismological Society of America*, 98:4, 1837–1848, 2008.

Barberopoulou, A., et al., “Long-Period Effects of the Denali Earthquake on Water Bodies in the Puget Lowland: Observations and Modeling,” *Bulletin of the Seismological Society of America*, 96:2, 519–535, 2006.

National Oceanic and Atmospheric Administration, “Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States,” NOAA Technical Report NWS 23, U.S. Department of Commerce, Washington, D.C., September 1979.

Saville, T. Jr., E. W. McClendon, A. L. Cochran, “Freeboard Allowances for Waters in Inland Reservoirs,” *Journal of Waterways & Harbors Division*, ASCE, May 1962.

South Texas Nuclear Operating Company, “South Texas Project Combined License Application,” Revision 0, Part 2, Final Safety Analysis Report, 2007.

South Texas Project Electric Generating Station, “STPEGS Updated Final Safety Analysis Report (UFSAR) for Units 1 & 2, Revision 13,” 2006.

U.S. Army Corps of Engineers, *Shore Protection Manual,* Third Edition, Coastal Engineering Research Center, Vicksburg, Mississippi, 1977.

U.S. Army Corps of Engineers, “Coastal Engineering Manual – Part II,” Engineering Manual 1110-2-1100, Coastal and Hydraulics Laboratory, Engineer Research and Development Center, Vicksburg, MS, 2008.

Wilson, B. W.,”Seiches,” *Advances in Hydroscience*, Vol. 8, Academic Press, New York, NY, 1972.

2.4S.9 Channel Diversions

2.4S.9.1 Introduction

This section of the FSAR addresses channel diversions. It evaluates plant and essential water supplies used to transport and impound water supplies to ensure that they will not be adversely affected by stream or channel diversions. The evaluation includes stream channel diversions away from the site (which may lead to a loss of safety-related water) and stream channel diversions toward the site (which may lead to flooding). In addition, in such an event, it must be ensured that alternate water supplies are available to safety-related equipment.

This SER section presents an evaluation of the following specific areas: (1) historical channel migration phenomena including cutoffs, subsidence, and uplift; (2) regional topographic evidence that suggests a future channel diversion may or may not occur (used in conjunction with evidence of historical diversions); (3) thermal causes of channel diversion, such as ice jams, which may result from downstream ice blockages that may lead to flooding from backwater or upstream ice blockages that can divert the flow of water away from the intake; (4) potential for forces on safety-related facilities or the blockage of water supplies resulting from channel migration-induced flooding (flooding not addressed by hydrometeorologically induced flooding scenarios in other sections); (5) potential of channel diversion from human-induced causes (i.e., land-use changes, diking, channelization, armoring, or failure of structures); (6) alternate water sources and operating procedures; (7) potential effects of seismic and nonseismic information on the postulated worst-case channel diversion scenario for the proposed plant site; (8) any additional information requirement prescribed in the “Contents of Application” sections of the applicable subparts of 10 CFR Part 52.

2.4S.9.2 Summary of Application

In Section 2.4S.9, the applicant describes site-specific information related to the channel diversions. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.18 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.18 Channel Division

COL License Information Item 2.18 requires the COL applicants to provide site-specific information related to channel diversion for the STP site. The following information addresses this subject.

2.4S.9.3 Regulatory Basis

The relevant requirements of the Commission regulations for the channel diversions, and the associated acceptance criteria, are described in Section 2.4.9 of NUREG–0800.

The applicable regulatory requirements for identifying and evaluating channel diversions are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave actions at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices

- RG 1.102, “Flood Protection for Nuclear Power Plants”

2.4S.9.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.9 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to the channel diversions. The staff’s technical review of this section includes an independent review of the applicant’s information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff’s evaluation of the technical information presented by the applicant in FSAR Section 2.4S.9.

<u>COL License Information Item</u>

- COL License Information Item 2.18 Channel Division

The staff reviewed the applicant’s supplemental information on channel diversions. The staff’s review of the applicant’s information is summarized below.

2.4S.9.4.1 Historical Channel Diversions

<u>Information Submitted by Applicant</u>

The applicant provides a review of the geology of the STP site vicinity, paleo-geology of the Colorado River Basin, current flow regulation of the Colorado River, and adjacent coastal areas. An examination of the stratigraphic evidence reveals that the Colorado River course near the STP site has maintained its present location for the last 550 years (STPNOC, 2007, FSAR Subsection 2.4S.9.2). The applicant concludes that changes in the present river course due to ice effects and surface faulting are considered unlikely (FSAR Subsections 2.4S.9.3 and 2.4S.9.4.1, respectively). From 1943 to 1973, the land surface in the vicinity of Bay City subsided more than 0.46 m (1.5 ft) because of groundwater withdrawals. However, the applicant finds that groundwater withdrawal rates are declining (FSAR Subsection 2.4S.9.4.2). Regulation by dams has minimized channel modification during floods (FSAR Subsections 2.4S.9.4.3 and 2.4S.9.5.2). Because Hurricane Carla caused no long-lived channel diversion, channel diversion due to coastal storms is considered unlikely (FSAR Subsection 2.4S.9.4.4).

The applicant states that sand and gravel mining in the Colorado River have taken place near Austin and subsequently the river has eroded new channel paths through abandoned pits in Travis and Colorado counties (FSAR Subsection 2.4S.9.5). However, the applicant states that severe bed degradation in the Lower Colorado River has not been observed. Dredging operations and channel stabilization in the Lower Colorado River have reportedly increased the bank full capacity of the river near the STP Units 3 and 4 site (FSAR Subsections 2.4S.9.5.2 and 2.4S.9.5). The applicant concludes that there is little likelihood of major channel diversions affecting STP Units 3 and 4 safety facilities (FSAR Subsection 2.4S.9.5).

NRC Staff's Technical Evaluation

The applicant's response to RAI 02.04.09-1 dated July 2, 2008 (ML081890239), states that the flood of 1935 had a peak discharge of almost 14,158 m^3/s (500,000 cfs). The applicant also states that the 1935 event was the last major flood to divert a significant flow of the Colorado River into the headwaters of the Tres Palacios Creek. The applicant argues that dams built upstream of the STP site in the Colorado River Basin provide flood control that has greatly reduced major flooding in lower portions of the basin.

The staff reviewed the applicant's response and determined that it is adequate. The applicant's response is consistent with the staff's independent review of historical floods in the Lower Colorado River Basin, as described in Section 2.4S.2 of this SER. The staff used this information when reviewing potential channel diversions of the Colorado River. The staff found the applicant's description of historical channel diversions acceptable. Therefore, RAI 02.04.09-1 is closed.

2.4S.9.4.2 Stratigraphic Evidence

Information Submitted by Applicant

The applicant states that stratigraphic evidence in the Colorado River and Caney Creek basins near the STP site suggests that the river has occupied its present course for more than 550 years. The most likely avulsion point on the Colorado River in the future is between Eagle Lake, Texas, and Wharton, Texas (Blum and Valastro Jr., 1994), where the modern Colorado River channel and the abandoned Caney Creek meander belt split within the same valley. Downstream of Wharton, the stream courses of the Colorado River and Caney Creek diverge until they reach the Gulf, separated by approximately 40 km (24.9 mi).

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR and cited references for the stratigraphic data. The staff found no particular evidence of a potential diversion of the

Colorado River. Furthermore, the Colorado River is currently highly regulated by upstream dams. Although the lower portions of the river have low relief, flood discharges into the channel near the STP site are greatly reduced since the construction of Mansfield Dam, making the diversion of the Colorado River unlikely.

2.4S.9.4.3 Ice Causes

Information Submitted by Applicant

The applicant considers channel diversion caused by ice jams unlikely on the Colorado River, because there are no historical records of any major rivers in Texas freezing.

NRC Staff's Technical Evaluation

The staff reviewed air temperature data near the STP site in Section 2.4S.8 of this SER and determined that ice formation is an unlikely event near the STP site. The staff also determined that no historical record of ice jam or ice dam formation on the Colorado River exists. The staff therefore concluded that ice is an unlikely cause of channel diversion near the STP site.

2.4S.9.4.4 Flooding of the Site Due to Channel Diversion

Information Submitted by Applicant

There are no reports of channel diversion upstream of the Balcones Escarpment near Austin, Texas. In the vicinity of the STP site the topography is flat, with an average dip of less than 1 degree in regional geologic units. The low slope also indicates a low probability of slope failure. There are no capable faults in the STP site region where surface faulting can occur to induce a slope failure leading to channel diversion.

Ground subsidence of 3.7 cm (0.12 ft) in the vicinity of Bay City, Texas, was measured between 1918 and 1951 (Hammond, Jr., 1969). Between 1943 and 1973, the land subsidence due to groundwater withdrawals in Matagorda County was 0.46 m (1.5 ft), which is attributed to increased groundwater use after 1940 (Ratzlaff, 1982). The Texas Water Development Board (2006) documented a decline in groundwater use in Matagorda County, from 47.6 million m^3 (38,554 ac-ft) in 1980 to 46.3 million m^3 (37,537 ac-ft) in 1990 and 17.8 million m^3 (14,413 ac-ft) in 1997. This reduction in the withdrawal of groundwater in Matagorda County should also minimize further subsidence.

A large flood or a series of large floods caused by upstream dam failures or significant changes in sea level could result in channel diversion in an unregulated Colorado River Basin. Because regulation in the basin since 1938 has helped to reduce the flood peak discharges, this mechanism of channel diversion is considered unlikely.

In 1961, Hurricane Carla partly obliterated the Matagorda peninsula, but the damage was soon repaired naturally by shoreline sediment migration and deposition (Hyde, 2001). The applicant concludes that hurricane effects are not considered a significant cause for channel diversion because even Hurricane Carla, which was a Category 5 hurricane, did not cause any channel diversions in the area (STPNOC, 2007).

Downstream of Austin, Texas, sand and gravel mining in the Colorado River have created pits. During flooding, the river has carved new paths through these abandoned pits at several locations in Travis and Colorado counties resulting in artificial cutoffs of historical meanders, and some localized downstream bank effects (Saunders, 2002). Although unconstrained gravel mining may lead to severe degradation downstream (Parker, 2008), none has been observed in the lower Colorado River. Therefore, the applicant concludes that the gravel mining effect will not contribute significantly to channel diversion near the STP site.

NRC Staff's Technical Evaluation

Because of low reliefs in the lower Colorado River Basin near the STP site, slope failures along the banks of the Colorado River are unlikely. There is also no potential for land subsidence or sand and gravel mining to divert the course of the Colorado River. Sections 2.4S.4 and 2.4S.5 respectively evaluate the effects of dam failures and hurricanes. The staff concluded that such events will not divert the Colorado River toward the STP site.

2.4S.9.4.5 Human-Induced Changes of Channel Diversion

Information Submitted by Applicant

A major log jam in the Colorado River had existed for a long time, from its earliest reference in 1690 to its first survey in 1824, where the log jam extended 74 km (46 mi) in length. The jam was finally removed in 1929 during a large flood assisted by earlier, manual removal efforts. The Colorado River delta, which was 182,105 m^2 (45 ac) in 1908, grew to 14 km^2 (3,470 ac) in 1933, 19.8 km^2 (4,890 ac) in 1936, 28.7 km^2 (7,098 ac) in 1941, and 29.1 km^2 (7,200 ac) in 1953. In 1936, a channel cut through the Matagorda peninsula relieved upstream flooding, and caused the river to discharge directly into the Gulf. The creation of upstream dams on the Colorado River has limited sediment delivery to the mouth and as a consequence, the delta has been receding.

During the flood of 1935, flow from the Colorado River was diverted into Tres Palacios Creek and the Tres Palacios Bay. The unregulated river may still be subject to such diversions (Wadsworth, 1966).

The USACE dredged the Colorado River from river kilometer 35 (river mile 22) to the Intracoastal Waterway to stabilize the river planform. The USACE deposited the dredged material along the river on both banks and enclosed by embankments. During this activity, the USACE also filled in the abandoned river channel north of the STP site in the vicinity of Selkirk Island. Because of these measures, the applicant considers a shifting of the river near the STP site unlikely.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information related to efforts to clear a long-existing log jam in the Colorado River. The USACE also periodically carries out maintenance in the Colorado River to keep the channel navigable. There are no major projects proposed on the Colorado River upstream and downstream of the STP site that may affect its present course. Based on the above review, the staff concluded that human-induced changes in the course of the Colorado River are minor, and the river will therefore not migrate from its present course.

2.4S.9.4.6 Potential of Future Channel Migration and Impact

Information Submitted by Applicant

Because of the presence of control structures upstream of the STP site on the Colorado River and the plan for stabilization measures on the lower Colorado River, the applicant concludes that channel diversion near the STP site is unlikely and will not produce a flood approaching the magnitude of the PMF discussed in FSAR Section 2.4S.2.

NRC Staff's Technical Evaluation

The discharge in the Colorado River near the STP site is highly regulated by upstream dams. There are no major projects proposed for the Colorado River upstream and downstream of the STP site. Therefore, the staff concluded that future channel migration of the Colorado River near the STP site is unlikely.

2.4S.9.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.9.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information to demonstrate that the characteristics of the site fall within the site parameters specified in the ABWR DCD, and no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description ensuring that the plant and essential water supplies will not be adversely affected. The staff reviewed the information provided and concluded, for the reasons given above, that the applicant has provided sufficient details to address COL License Information Item 2.18. Therefore, the staff concluded that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site.

2.4S.9.7 References

Blum, M. D. and S. Valastro, Jr., "Late Quaternary Sedimentation, Lower Colorado River, Gulf Coastal Plain of Texas," *Geological Society of America Bulletin*, 106:1002–1016, 1994.

Hammond, Jr., W.W., "Ground-Water Resources of Matagorda County, Texas," Texas Water Development Board Report 91, March 1969.

Hyde, H. W., "Soil Survey of Matagorda County, Texas," United States Department of Agriculture, Natural Resources Conservation Service, 2001.

Mace, R.E., et al., eds., "Aquifers of the Gulf Coast of Texas," Texas Water Development Board Report 365, February 2006.

Parker, G., “Transport of Gravel and Sediment Mixtures,” Chapter 3, in ASCE Manual 110, *Sedimentation Engineering: Processes, Measurements, Modeling, and Practice*, Garcia, M., ed., American Society of Civil Engineers, p. 1150, 2008.

Ratzlaff, K.W., “Land-Surface Subsidence in the Texas Coastal Region,” Texas Water Development Board Report 272, November 1982.

Saunders, G.P., “Impacts of Sand and Gravel Mining on Physical Habitat of the Colorado River and Tributaries, Central Texas,” Transactions of the Gulf Coast Association of Geological Societies, pp. 883–890, 2002.

South Texas Nuclear Operating Company, “South Texas Project Combined License Application," Revision 0, Part 2,” Final Safety Analysis Report, 2007.

Wadsworth, A.H., “Historical Deltation of the Colorado River, Texas,” in *Deltas in Their Geologic Framework*, Shirley, M-L. ed., Houston Geological Society, 1966.

2.4S.10 Flooding-Protection Requirements

2.4S.10.1 Introduction

This section of the FSAR addresses the design bases required to ensure that safety-related facilities will be capable of surviving all design-basis flood conditions, and those of structures and components required for protection of safety-related facilities. This section also describes various types of flood protection used and the emergency procedures to be implemented where applicable.

This SER section provides an evaluation of the following specific areas: (1) safety-related facilities exposed to flooding; (2) type of flood protection (e.g., "hardened facilities," flood doors, bulkheads, etc.) provided for the SSCs exposed to flooding; (3) emergency procedures needed to implement flood-protection activities and warning times available for their implementation reviewed by the organization responsible for reviewing issues related to plant emergency procedures; (4) potential effects of seismic and non-seismic information on the postulated flood-protection for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.10.2 Summary of Application

In Section 2.4S.10, the applicant addresses the need for site-specific information on flood-protection requirements. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.19 identified in DCD Tier 2, Revision 4, Section 2.3.

Tier 1 Departure

- STP DEP T1 5.0-1 Site Parameters

The applicant identifies the departure of the site-specific maximum flood level from the ABWR DCD standard plant site design maximum flood level parameter, as described in DCD Tier 1, Table 5.

COL License Information Item

- COL License Information Item 2.19 Flooding Protection Requirements

COL License Information Item 2.19 requires the COL applicants to provide site-specific information related to flood protection for the STP site.

2.4S.10.3 Regulatory Basis

The relevant requirements of the Commission regulations for flooding-protection requirements, and the associated acceptance criteria, are in Section 2.4.10 of NUREG–0800.

The applicable regulatory requirements for identifying and evaluating flood-protection requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to flood levels and wave action at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants"

- RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"

In addition, in accordance with Section VIII, "Processes for Changes and Departures," of "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies one Tier 1 departure requiring prior NRC approval. This departure is subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4. .

2.4S.10.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.10 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the application addresses the relevant information related to the flooding-protection requirements. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.10.

<u>Tier 1 Departure</u>

- STP DEP T1 5.0-1 Site Parameters

The staff's evaluation of this departure as it relates to design-basis flood and protection of safety-related systems is discussed below.

COL License Information Item

- COL License Information Item 2.19 Flooding Protection Requirements

The staff reviewed the applicant's supplemental information on flooding-protection requirements. The staff's review of the application is summarized below.

Information Submitted by Applicant

The applicant states in FSAR Section 2.4S.2 that the design-basis floodwater elevation in the STP Units 3 and 4 power block area is 12.2 m (40 ft) MSL, which is higher than the proposed site grade in the power block area that ranges from 9.8 to 11.2 m (32 to 36.6 ft) MSL. Therefore, the applicant states that all safety-related SSCs of the proposed STP Units 3 and 4 will require flood protection to the design-basis floodwater elevation of 12.2 m (40 ft) MSL, and identifies this in Departure STP DEP T1 5.0-1.

The applicant, under COL Information Item 2.19, identifies safety-related SSCs requiring flood protection. The applicant states that safety-related SSCs for STP Units 3 and 4 include the reactor buildings, control buildings, the UHS water-storage basins, the UHS cooling towers, and RSW pump houses. The applicant adds that these facilities are designed to withstand the combination of flooding conditions and wave runup, including both static and dynamic flooding forces associated with the flooding events, and that the foundations of these facilities are deep enough to withstand the erosive forces resulting from the main cooling reservoir embankment breach.

The safety-related facilities must remain free from flooding and intrusion of water into areas that contain safety-related equipment. The applicant states that all safety-related facilities in the power block area are watertight below 12.2 m (40 ft) MSL. The applicant states that all watertight doors and hatches open outward and are normally closed position under administrative controls. The applicant adds that all ventilation openings are located above 12.2 m (40 ft) MSL, and that the UHS and RSW pump houses are designed to be watertight below 15 m (50 ft) MSL.

The staff issued Supplemental RAI 02.04.05-11 requesting the applicant to provide additional information regarding the PMSS estimation at the STP site and a possible failure of the main cooling reservoir northern embankment due to erosive action of PMSS waters. The following parts of RAI 02.04.05-11 are relevant to this section of the SER: (8) detailed description of various methods used to estimate current velocities during a PMSS event; (9) a detailed description and justification of simplifying assumptions; (10) conservatively selected current velocities and durations for which these currents will affect the main cooling reservoir northern embankment; and (11) justification, including relevant citations, for the ability of the grass-lined outer face of the main cooling reservoir northern embankment to withstand the current velocities without erosion severe enough to cause an embankment breach. The applicant responded in a letter dated November 22, 2010 (ML103330369).

The applicant describes the erosion protection features of the main cooling reservoir northern embankment. The applicant states that the outer face of the main cooling reservoir northern embankment is grass-lined with a slope of 3 horizontal to 1 vertical. The applicant states that the ADCIRC prediction of the PMSS water surface elevation at the STP site, including wave runup, is 8.9 m (29.3 ft) MSL, which is lower than the grade elevation of 10.4 m (34 ft) MSL at the northern face of the main cooling reservoir northern embankment. Therefore, the applicant concludes that failure of the main cooling reservoir northern embankment from the sloshing and erosive action of PMSS waters is not a credible event. The applicant also describes a more conservative scenario where the PMSS was estimated using the SLOSH model. The applicant states that in this conservative scenario, SLOSH predicted a stillwater storm surge water surface elevation of 11.7 m (38.5 ft) MSL and the coincident wind-wave action would raise the storm surge water surface elevation to 12.7 m (41.8 ft) MSL. The applicant states that the time history of this conservative scenario showed that the PMSS water surface elevation would be at 10.4 m (34 ft) MSL (i.e., at site grade) for 80 minutes; at or above 11 m (36 ft) MSL for 50 minutes; and at or above 11.6 m (38 ft) MSL for 25 minutes. The applicant states that significant erosion of the grass-lined north face of the main cooling reservoir northern embankment would not occur during this short amount of time, because a grass surface works well for short-term exposure as plant roots keep soil particles bound together to create a flexible system that deforms without tearing. The applicant also states that the flood-protection levee for Texas City survived a sustained surge and wave attack during Hurricane Ike for many hours without a breach (USACE, 2009). The applicant notes that the main cooling reservoir embankment is similar to but much larger than typical hurricane surge-protection levees that have mostly withstood major hurricanes in the past.

In response to RAI 02.04.05-11 part (8), the applicant states that water will flow past the main cooling reservoir northern embankment under the conservative PMSS scenario predicted by the SLOSH. The applicant notes that the SLOSH does not output current velocities, but they can be estimated using (1) the area around the STP Units 3 and 4 that experiences the PMSS and matching the volume of water that fills and drains through this area during the PMSS event; (2) using Manning's n and a friction slope estimated by change in water surface elevations; and (3) tracking the PMSS wave-front past the site. The applicant uses all three methods to estimate current velocities during the PMSS event.

In response to RAI 02.04.05-11 part (9), the applicant states that a storm surge that would exceed the STP Units 3 and 4 site grade elevation of 10.4 m (34 ft) MSL is not a credible event. The applicant notes that ADCIRC predictions resulted in a PMSS water surface elevation of 8.9 m (29.3 ft) MSL, which is significantly less than the STP Units 3 and 4 site grade elevation. The applicant also states conservative predictions from the SLOSH resulted in a PMSS water surface elevation that would inundate only a small portion of the main cooling reservoir northern embankment for a short duration. The applicant concludes that any erosion at the base of the main cooling reservoir northern embankment would not threaten a failure.

In response to RAI 02.04.05-11 part (10), the applicant states that the maximum current velocities estimated using the three methods listed above are 3.5 m/s (11.6 fps), 0.9 m/s (3.1 fps), and 1.9 to 4 m/s (6.2 to 13.2 fps), respectively. The applicant also states that the PMSS flow past the main cooling reservoir northern embankment would occur for a maximum duration of 80 minutes.

In response to RAI 02.04.05-11 part (11), the applicant states that the USACE recommends a design velocity of 1.5 to 2.4 m/s (5 to 8 fps) for stable grass-lined flood channels. The applicant states that the grass-lined main cooling reservoir embankment can be expected to sustain a short exposure to current velocities slightly higher than those assumed in the design of flood channels that have a design life of several decades and would likely be subject to flow durations considerably longer than 80 minutes. The applicant concludes that erosion of the main cooling reservoir northern embankment is unlikely.

NRC Staff's Technical Evaluation

Subsection C.I.2.4.10 of RG 1.206 specifies that "the applicant should describe the static and dynamic consequences of all types of flooding on each pertinent safety-related facility." Additionally, Subsection C.I.2.4.14 of RG 1.206 states that "[i]f the applicant will use emergency procedures ... appropriate water levels and lead times available should be provided." Subsection C.I.2.4.14 also states that "the applicant should develop specific details on ... the amount of time available to initiate and complete emergency procedures." To meet the above requirements, the staff issued RAI 02.04.10-1 requesting the applicant to provide severe flood levels in addition to other flood parameters, such as flow velocity and duration (beginning, peak, and end) of inundation important for the design of safety-related SSCs and the preparation of emergency procedures.

As part of the review of COL License Information Item 2.19, the staff asked the applicant to discuss the potential effects on the safety-related facilities of the composition of the flood wave (essentially a mudflow), with respect to the sediment (generated from the gradual breach of the main cooling reservoir embankment) carried with the flow, including dynamic and impact forces. The staff asked the applicant to discuss the conservatism of this case compared to the case presented in the FSAR. The staff postulated that a failure of the main cooling reservoir embankment breach could result in an accumulation of a large amount of bank material at the plant site. The staff asked the applicant to discuss the effects of the settlement of these bank materials around the safety-related structures; the necessary shutdown or operation procedures of the plant after the postulated main cooling reservoir northern embankment failure; and how these effects, if significant, will be addressed in FSAR Section 2.4.14, "Technical Specifications and Emergency Operations Requirements."

The applicant's response to RAI 02.04.10-1 dated November 13, 2008 (ML083250480), states that the entrance-level slab elevation of STP Units 3 and 4 safety-related SSCs is 10.7 m (35 ft) MSL. The applicant also states that the STP Units 3 and 4 site will experience a floodwater surface elevation exceeding 10.7 m (35 ft) MSL under two scenarios: (1) the flood in the power block area under the effects of a local PMP event, and (2) the flood resulting from a postulated breach of the main cooling reservoir northern embankment. Based on Subsection C.I.2.4.14 of RG 1.206, the staff finds this response reasonable acceptable and considers RAI 02.04.10-1 closed.

The applicant reports that using HEC-RAS software to estimate the local PMP flows, the average estimates of cross-sectional velocities within the power block area were between 0.03 to 0.21 m/s (0.1 and 0.7 fps) in the West Channel, which will be located west of STP Unit 4. The applicant states that the average cross-sectional velocities in the East Channel, which will be located east of the STP Unit 3 power block, were between 0.06 to 0.37 m/s (0.2 and 1.2 fps). The applicant reports that the estimated total duration of runoff during the local PMP event was approximately 7 hours in both the West and the East Channels. The applicant also states that the duration of discharges exceeding 28.3 m^3/s (1,000 cfs) was less (1 hour in both the West and the East Channels). The applicant notes that the local PMP event is a slow-moving event that allows the plant operators sufficient time to take action.

In the letter dated January 28, 2009 (ML091880126), the applicant reports that the flood resulting from the main cooling reservoir northern embankment breach was simulated using RMA2, which is a two-dimensional, depth-averaged, hydrodynamic model developed by the USACE (2005). Section 2.4S.4 of this SER describes the staff's review of the flood, erosion, and sedimentation and sediment transport following the postulated breach of the main cooling reservoir northern embankment. The applicant has proposed a site characteristic of 12.2 m (40 ft) MSL for the highest floodwater surface elevation at the STP Units 3 and 4 site.

The applicant also states that the sediment-laden floodwaters will produce a greater force on SSCs compared to non-sediment-laden waters. The applicant reports that the maximum simulated flow velocity is approximately 1.4 m/s (4.7 fps), and the maximum simulated sediment concentration of the flow is 23 kg/m^3. The applicant estimates that the maximum drag force on the projected submerged area of the SSCs would be 214.8 kg/m^2 (44 lb/ft^2).

In Section 2.4S.4 of this SER, the staff postulated a combination of events that could be triggered from erosion of the toe of the main cooling reservoir northern embankment during the PMSS event. Because the applicant had not yet provided an analysis to show whether this is a plausible event, the staff did not confirm the design-basis flood elevation of 12 m (40 ft) MSL reported in FSAR Section 2.4S.4 and the drag forces on SSCs reported above. This issue was tracked as Open Item 2.4.10-1 in the SER with open items.

The applicant responded to RAI 02.04.05-11 parts (8) through (11), as described above. The staff reviewed the applicant's submittal. As described in Section 2.4S.5.4 of this SER, the staff determined that the applicant has performed a reasonable and conservative site-specific estimate of the PMSS. The staff agreed with the applicant's conclusion that the maximum PMSS water surface elevation accounting for a wind setup effect would not exceed 8.9 m (29.3 ft) MSL. As described in Section 2.4S.5.4 above, the staff concluded that the maximum PMSS water surface elevation at the STP Units 3 and 4 site accounting for a wind setup and runup would not exceed 9.1 m (30 ft) MSL, 1.2 to 1.8 m (4 to 6 ft) below the STP Units 3 and 4 site grade of 10.4 to 11 m (34 to 36 ft) MSL. The applicant states in FSAR Subsections 2.4S.4.2.2.2.2 and 2.4S.4.2.2.2.4.1 that the terrain immediately downstream of a service road running along the toe of the exterior slope of the main cooling reservoir northern embankment acts as a control against the development of a breach. The applicant states that the terrain elevation at this location is 8.8 m (29 ft) MSL. Because the maximum PMSS water surface elevation including the wind setup and runup effects is 9.1 m (30 ft) MSL, the staff concluded that the lower reach of the toe of the main cooling reservoir northern embankment would experience currents during the PMSS event. The slope of the main cooling reservoir northern embankment at this location is 6 horizontal to 1 vertical (Figure 10 in the applicant's response to RAI 02.04.05-11, ML103330369). Because of the gentle slope and relatively small area of the toe of the main cooling reservoir northern embankment that would be inundated during the PMSS event, the staff concluded that it is unlikely that PMSS currents would cause significant damage to the toe of the northern embankment. For the main cooling reservoir embankment to fail, the erosive action of the PMSS current would have to erode the toe to such an extent that (1) a pipe would form extending to the interior face of the embankment; or (2) an extensive sliding surface would form extending from the downstream to near the upstream face of the embankment. Because the toe of the main cooling reservoir northern embankment is inundated only with a depth of 0.3 m (1 ft) near the exterior end of the embankment, the staff concluded that such a failure mechanism is unlikely. During the PMSS event, the STP Units 3 and 4 power block with a grade elevation of 10.4 to 11 m (34 to 36 ft) MSL would remain unaffected, because the PMSS water surface elevation would not exceed 9.1 m (30 ft) MSL. Therefore, the staff concluded that even in the unlikely scenario that the main cooling reservoir northern embankment were to fail because of erosive action of PMSS currents, the resulting flood would be similar to and not more severe than that analyzed in Section 2.4S.4 of the FSAR, which the staff reviewed in Section 2.4S.4 of this SER. Therefore, the staff determined that the characteristics of the design-basis flood related to Departure STP DEP T1 5.0-1 and the corresponding drag forces on safety-related SSCs are as described in FSAR Section 2.4S.4 and reviewed by the staff in Section 2.4S.4 of this SER.

Because of the reasons stated above, the staff determined that further characterization of a PMSS-induced main cooling reservoir northern embankment failure is not warranted. Therefore, Open Item 2.4.10-1 is closed.

2.4S.10.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.10.6 Conclusion

NRC staff reviewed the application and confirmed that the applicant has addressed the information demonstrating that the characteristics of the site fall within the site parameters specified in the ABWR DCD, and no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information relative to the flood protection measures important to the design and siting of this plant. The staff found that the applicant has considered the appropriate site phenomena in establishing the flood protection measures for SSCs. The staff reviewed the applicant's information and, for the reasons stated above, concluded that the applicant, as documented in Section 2.4S.10 of this SER, has provided sufficient details about the site description to allow the staff to evaluate whether the applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site. The information addressing COL License Information Item 2.19 is adequate and acceptable. The characteristics of the design-basis flood related to Departure STP DEP T1 5.0-1 are described in Section 2.4S.4 of this report.

2.4S.10.7 References

GAO, "Hurricane Protection: Statutory and Regulatory Framework for Levee Maintenance and Emergency Response for the Lake Pontchartrain Project," GAO-06-322T, Washington, DC, 2006.

U.S. Army Corps of Engineers, "User's Guide to SED2D WES," Version 4.5. Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, Vicksburg, MS, 2003.

U.S. Army Corps of Engineers, "User's Guide to RMA2 WES," Version 4.5, Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, Vicksburg, MS, 2005.

2.4S.11 Low Water Considerations

2.4S.11.1 Introduction

This section of the FSAR addresses natural events that may reduce or limit the available safety-related cooling-water supply. The applicant ensures that an adequate water supply will exist to shut down the plant under conditions requiring safety-related cooling.

This SER section provides an evaluation of the following specific areas: (1) low-water conditions due to the worst drought considered reasonably possible in the region; (2) the effects of low water surface elevations caused by various hydrometeorological events and a potential blockage of intakes by sediment, debris, littoral drift, and ice because they can affect the safety-related water supply; (3) the effects of low water on the intake structure and pump design bases in relation to the events described in FSAR Sections 2.4S.7, 2.4S.8, 2.4S.9, and 2.4S.11, which consider the range of water supply required by the plant (including minimum operating and shutdown flows during anticipated operational occurrences and emergency conditions) compared with availability (considering the capability of the UHS to provide adequate cooling water under conditions requiring safety-related cooling); (4) the use limitations imposed or under discussion by Federal, State, or local agencies authorizing the use of the water; (5) the potential effects of seismic and non-seismic information on the postulated worst-case low-water scenario for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.11.2 Summary of Application

In Section 2.4S.11, the applicant addresses the impacts of low water on safety-related water supply. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.20, identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addresses the information as follows:

COL License Information Item

- COL License Information Item 2.20 Cooling-Water Supply

COL License Information Item 2.20 requires the COL applicants to provide site-specific information related to the cooling-water supply for the STP site. The following information addresses this subject.

2.4S.11.3 Regulatory Basis

The relevant requirements of the Commission regulations for low water considerations, and the associated acceptance criteria, are in Section 2.4.11 of NUREG–0800.

The applicable regulatory requirements for identifying the effects of low water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices

2.4S.11.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.11 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the low-water considerations. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.11.

COL License Information Item

- COL License Information Item 2.20 Cooling-Water Supply

The staff issued RAI 02.04.11-1 requesting the applicant to provide details to support the following statement in FSAR Subsection 2.4S.11.6 or to delete the statement if it is not relevant here: "The potential effects of all site-related proximity, seismic, and non-seismic information on the postulated worst-case low-flow scenario for the proposed plant site have been considered in establishing the design basis."

In a letter dated June 26, 2008 (ML081970231), the applicant's response to RAI 02.04.11-1 states that the statement is not relevant to FSAR Section 2.4S.11. The applicant has removed the statement from the FSAR. The staff is satisfied with this change and therefore, RAI 02.04.11-1 is considered closed.

2.4S.11.4.1 Low Flow in Rivers and Streams

Information Submitted by Applicant

The STP Units 3 and 4 site is located on the west bank of the Colorado River at river kilometer 23.5 (river mile 14.6). Tidal influence reaches upstream to river kilometer 35.4 (river mile 22). An inflatable dam 1.6 km (1 mi) downstream from Bay City and immediately upstream from the USGS gauge station at Bay City (08162500) is used to maintain water quality for irrigation withdrawals. Discharge data at this station are available from 1948 but are affected by the presence of upstream dams.

Zero daily discharge was recorded 13 times from 1951 to 1956. During June and July of 1967, irrigation withdrawals reduced the downstream flow to less than 0.028 m^3/s (1 cfs) for 58 days. The recorded minimum 1-day and 7-day low flows are 0 and 0.014 m^3/s (0 and 0.5 cfs), respectively.

The primary source of makeup water to the UHS water-storage basins will be onsite groundwater wells that are unaffected by low-flow conditions in the Colorado River. The main cooling reservoir will provide a backup source of UHS makeup water for the UHS.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown. Also, the staff confirmed that the Colorado River water will not be used as a source of UHS makeup. Therefore, the staff determined that low flow in river and streams will not affect the safe operation of STP Units 3 and 4.

2.4S.11.4.2 Low Water Resulting from Surges, Seiches, or Tsunamis

Information Submitted by Applicant

The applicant proposes groundwater wells as the primary source of makeup water to the UHS water-storage basin. Groundwater conditions are not expected to be affected by low water resulting from surges, seiches, or tsunamis. Formation of ice jams or ice dams near the STP site is unlikely based on historical air and water temperature observations near the STP site. Therefore, the applicant concludes that low water resulting from ice-induced causes is unlikely.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe-shutdown conditions for 30 days, without requiring any makeup or blowdown. Therefore, the staff determined that low water resulting from surges, seiches, or tsunamis will not affect the safety of STP Units 3 and 4.

2.4S.11.4.3 Historical Low Water

Information Submitted by Applicant

The most severe drought event on record, based on observations from 1898 through 2004, is the 10-year drought that spanned from May 1947 to April 1957.

The inflatable dam below Bay City, which was installed in 1963, regulates low flow in the Colorado River. During extremely low-flow conditions in the Colorado River, the river water surface elevation near the RMPF is expected to be approximately equal to the tidal elevation to prevent saltwater intrusion.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. The staff's review determined that the applicant has provided sufficient information and the description of the historical low water is adequate and acceptable.

2.4S.11.4.4 Future Controls

Information Submitted by Applicant

The safety-related systems of STP Units 3 and 4, including the UHS, do not depend on the Colorado River or the main cooling reservoir as water sources directly. Ground water is the primary source of makeup water to the UHS basins. The units will be shut down when the water surface elevation in the main cooling reservoir drops below 7.8 m (25.5 ft) MSL. At this elevation, the main cooling reservoir contains 47.1 million m^3 (38,150 ac-ft) of water, which exceeds the 30-day UHS makeup water requirements.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

Based on this information, the staff determined that the development of any future controls on the Colorado River water or on groundwater supplies will not have an adverse effect on the safety-related water held in the dedicated UHS water-storage basins for STP Units 3 and 4.

2.4S.11.4.5 Plant Requirements

Information Submitted by Applicant

The RSW and the UHS systems provide essential cooling during normal operation, normal shutdown, emergency shutdown, testing, and loss of preferred power while maintaining the temperature of the UHS water basin at or below 35 °C (95°F). The water-storage basins for the UHS (one each for STP Units 3 and 4) are designed with sufficient capacity to provide cooling during shutdown and cooldown and to maintain safe-shutdown conditions for 30 days, without the need for any makeup or blowdown. Water from the UHS basins is lost because of natural and forced evaporation, drift, seepage, and blowdown. The primary sources of makeup water to the UHS basins are site wells. The main cooling reservoir is the secondary source of makeup water provided to the basins through the turbine service-water system.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

Based on this information, the staff determined from the applicant's information in the FSAR that the primary sources of makeup water to the UHS water-storage basins are site wells. The main cooling reservoir, via the turbine service-water system, will be used as the secondary source of makeup water to the UHS water-storage tanks.

2.4S.11.4.6 Heat Sink Dependability Requirements

Information Submitted by Applicant

The UHS water-storage basins are sized to hold sufficient water to provide cooling and to maintain a safe shutdown following a design-basis accident for 30 days, without any reliance on makeup water.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's information in the FSAR. There is a separate UHS for each of STP Units 3 and 4 that is configured with a dedicated, partially buried water-storage basin sized to hold sufficient water to provide cooling following a design-basis accident and to maintain safe shutdown conditions for 30 days, without requiring any makeup or blowdown.

2.4S.11.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.11.6 Conclusion

The staff reviewed the applicant's information in the FSAR and supplemented that with observations from the staff's site audit and other publicly available data sources. The STP Units 3 and Unit 4 will each have an engineered, partially buried water-storage tank. These UHS water-storage tanks will be designed to hold sufficient water to provide cooling following a design-basis accident and to maintain a safe shutdown for a period of 30 days, without makeup or blowdown. The makeup water for the two UHS storage basins will come from site wells, which are the primary source, and from the main cooling reservoir, which is the secondary source. The staff determined that low-water events in the vicinity of the STP Units 3 and 4 site will not affect their safe operation. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant presents and substantiates information relative to the low-water effects important to the design and siting of this plant. The staff reviewed the available information and concluded, for the reasons given above, that the identification and consideration of the potential for low-water conditions are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to determining the acceptability of the site.

Therefore, the staff found that the applicant has considered the appropriate site phenomena in establishing the design bases for SSCs important to safety. The staff accepted the methodologies used to determine the potential for low-water conditions, as reflected in these design bases and documented in SERs for previous licensing actions. Accordingly, the staff concluded that the use of these methodologies results in design bases containing a margin sufficient for the limited accuracy, quantity, and period of time in which the data were accumulated. The staff concluded that the identified design bases meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing COL License Information Item 2.20 is adequate and acceptable.

2.4S.11.7 References

South Texas Nuclear Operating Company, “South Texas Project Combined License Application,” Revision 0, Part 2, Final Safety Analysis Report, 2007.

2.4S.12 Groundwater

2.4S.12.1 Introduction

This section of the FSAR describes the hydrogeological characteristics of the site. The most significant objective of groundwater investigations and monitoring at this site is to evaluate the effects of groundwater on safety-related plant facilities. The evaluation is performed to ensure that the maximum groundwater elevation remains below the DCD site parameter value. The other significant objectives are to examine whether the groundwater provides any safety-related water supply, determine whether dewatering systems are required to maintain groundwater elevation below the required level, measure characteristics and properties of the site needed to develop a conceptual site model of groundwater movement, and estimate the direction and velocity of movement of potential radioactive contaminants.

This SER section provides a review of the following specific areas: (1) description and onsite groundwater use, (2) groundwater source, (3) subsurface pathways, (4) monitoring and safeguard requirements, and (5) site characteristics for subsurface hydrostatic loading.

2.4S.12.2 Summary of Application

In Section 2.4S.12, the applicant addresses groundwater conditions in terms of influences on structures and water supply. In addition, the applicant provides site-specific supplemental information to address COL License Information Item 2.32 identified in DCD Tier 2, Revision 4, Section 2.3.

COL License Information Item

- COL License Information Item 2.32 Effect of Groundwater

This COL license information item directs the applicant to provide site-specific information that addresses groundwater conditions in terms of influences on structures and water supply. Specifically, the DCD states that COL applicants (1) "will analyze the groundwater condition for the specific site," and (2) "will evaluate the effect of groundwater on such site geotechnical properties as total and effective unit weights, cohesion and angle of internal friction, and dynamic soil properties." This section of the FSAR addresses the first of these subtopics, and FSAR Section 2.5.4 addresses the second subtopic.

2.4S.12.3 Regulatory Basis

The relevant requirements of the Commission regulations for groundwater, and the associated acceptance criteria, are described in Section 2.4.12 of NUREG–0800.

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4S.12.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.12 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the groundwater. The staff's technical review of this section includes an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.12.

COL License Information Items

- COL License Information Item 2.32 Effect of Groundwater

The staff reviewed the applicant's supplemental response on groundwater. The staff's review of the application is summarized below.

The staff's discussion of groundwater characteristics is organized into the following technical areas. Unresolved RAIs and open items are described where appropriate within these areas.

2.4S.12.4.1 Regional Hydrogeologic Description

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1, the applicant describes the geologic formations, regional and local groundwater aquifers, aquifer recharge and discharge regions, and onsite groundwater use. The applicant formulates a hydrogeologic conceptual model of the STP site using four different data sources that include the following:

- a desktop study of the regional groundwater system derived from State, Federal, and other sources of information

- a review of STP Units 1 and 2 documentation with regard to groundwater

- the collection of site-specific geotechnical, geologic, and hydrogeologic data for STP Units 3 and 4

- the evaluation of site-specific hydrogeology through regional data and information

The applicant considers site-specific STP Units 3 and 4 data in the context of site-specific STP Units 1 and 2 data and regional data to formulate the conceptual model for the STP site, with a focus on the proposed location for STP Units 3 and 4.

In FSAR Subsection 2.4S.12.1.1, the applicant describes the STP site as being in Matagorda County, Texas, and within the Gulf Coastal Plains physiographic province of the Coastal Prairies sub-province. The applicant describes the Coastal Prairies sub-province as follows. The Coastal Prairies sub-province is a broad band paralleling the Texas Gulf Coast (Ryder, 1996). The sub-province is characterized by relatively flat topography ranging from sea level at the coast to 91 m (300 ft) MSL along the northern and western inland boundaries of the sub-province. Underlying the STP site is a wedge of southeasterly dipping sedimentary deposits ranging in age from Holocene (i.e., 10,000 years ago to present) through Oligocene (i.e., 33.9 million to 23 million years before present).

In FSAR Subsection 2.4S.12.1.2, the applicant describes the Coastal Lowlands Aquifer System underlying the STP site. Within Texas, the term Gulf Coast Aquifer is used to describe this aquifer system (Mace et al., 2006). Numerous local aquifers are found in the thick sequence of alternating and interfingering beds of clay, silt, sand, and gravel. Ground water ranging in quality from fresh to saline is found in these sediments. Three depositional environments are evident: continental (alluvial plain); transitional (delta, lagoon, beach); and marine (continental shelf). Oscillations of the ancient shorelines have resulted in overlapping mixtures of sediments. The Texas nomenclature shown by the applicant in FSAR Figure 2.4S.12-5 is used to describe the aquifer system underlying the site. The common regional hydrogeologic unit names are as follows:

- Chicot Aquifer
- Evangeline Aquifer
- Burkeville Confining Unit
- Jasper Aquifer
- Catahoula Confining Unit
- Vicksburg-Jackson Confining Unit.

The applicant describes the Gulf Coast Aquifer (referred to here as the regional aquifer) as extending to either its contact with the top of the Vicksburg-Jackson Confining Unit or the depth where groundwater contains a total dissolved solids (TDS) concentration greater than 10,000 mg/L (0.78 lb/ft^3) [Ryder, 1996]). The regional aquifer system is recharged by precipitation falling on the aquifer outcrop areas along the northern and western boundaries of the physiographic province. It discharges through evapotranspiration, the loss of water as the base flow into streams, discharge into the Gulf of Mexico, and water pumped from groundwater wells.

In FSAR Subsection 2.4S.12.1.3, the applicant describes the hydrogeology of the Chicot Aquifer underlying Matagorda County. In this vicinity, the aquifer extends from the land surface to a depth of more than 304.8 m (1,000 ft). The stratigraphic units that compose the Chicot Aquifer in this vicinity, from the land surface downward, include the Holocene alluvium of the river valley; the Pleistocene age (i.e., from 1.8 M years ago to approximately 10,000 years ago) Beaumont, Montgomery, and Bentley Formations; and the Willis Sand. In general, the groundwater flows toward the south and southeast and the Gulf of Mexico. However, river channel incisions can act as localized areas of recharge or discharge and result in varied groundwater flow directions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsections 2.4S.12.1.1, 2.4S.12.1.2, and 2.4S.12.1.3. The staff's review confirmed that the applicant has addressed relevant information. In its review of the application, the staff found the applicant's information comparable to that in documents on the hydrology and aquifers of the region by the USGS (Ryder, 1996; Ryder and Ardis, 2002); the TWDB (Hammond, 1969; Mace et al., 2006); and the LCRA (Young et al., 2007). Based on the above information, the staff concluded that the applicant's descriptions of the regional hydrogeologic setting, regional groundwater aquifers, and the local hydrogeology are accurate.

2.4S.12.4.2 Site-Specific Hydrogeology

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.4 and the applicant's proposed revision in the responses to RAI 02.04.12-28 dated November 23, 2009 (ML093310392), the applicant describes the Chicot Aquifer underlying the STP site as an aquifer divided into two aquifer units: the Shallow Aquifer and the Deep Aquifer. The Shallow Aquifer is recharged a few miles north of the STP site and discharges into the alluvial material of the Colorado River east of the site and into groundwater wells. The Deep Aquifer is recharged farther north in Wharton County at aquifer outcrops and discharges to groundwater wells, the Colorado River estuary, and Matagorda Bay. In general, the groundwater quality of the Deep Aquifer is superior to that of the Shallow Aquifer and consequently, the Deep Aquifer is the primary groundwater production zone.

The applicant notes that the Shallow Aquifer can be subdivided into an Upper Shallow Aquifer and a Lower Shallow Aquifer. The applicant states that it completed 28 groundwater observation wells in the Upper and Lower Shallow Aquifer during initial site characterization activities and completed an additional 26 observation wells in 13 well clusters during July and August of 2008 (ML092710096), as described in RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The initial 28 observation wells were designed and located to supplement the existing STP network and provide a basis for estimating hydraulic gradients and determine the plausible current and future groundwater flow directions. Among the wells, several are designed to provide vertical hydraulic gradient data. The additional 26 wells supplement the aquifer data and better resolve alternative pathways originating in the vicinity of the main cooling reservoir and the proposed power block. The applicant also collected piezometric data monthly from December 2006 through 2007 and quarterly throughout 2008. Data collected since September 2008 include all 54 wells.

The applicant states that site characterization data collected for the proposed STP Units 3 and 4 confirmed and expanded the understanding of the aquifers that underlie the STP site. FSAR Table 2.4S12-14 and its proposed changes in RAI responses (ML101390277 and ML093310392) is reproduced here as Tables 2.4S.12-1 and 2.4S.12-2 in this SER to show the representative thickness of the hydrogeologic units and the properties of confining layers and aquifers in the STP hydrogeologic profile. In sequence from the ground surface to the deepest aquifer affected directly by the plant operation are the following units and thicknesses:

- Upper Shallow Aquifer confining layer, 6.1 m (20 ft)
- Upper Shallow Aquifer, 7.6 m (25 ft)
- Lower Shallow Aquifer confining layer, 6.1 m (20 ft)
- Lower Shallow Aquifer, 12.2 m (40 ft)

- Deep Aquifer confining layer, 30.5 m (100 ft)
- Deep Aquifer, 152.4 m (500 ft)

Currently, there are five completed STP production wells; the deepest reach 213 m (700 ft) below ground surface (BGS). There is some communication between the Upper and Lower Shallow Aquifers, but there appears to be little communication between the Shallow and Deep Aquifers.

The applicant acknowledges that the main cooling reservoir influences the hydraulic head within the Upper Shallow Aquifer; however, the applicant has concluded based on recently collected piezometric data presented in the response to RAIs dated September 21, 2009 (ML092710096) and November 23, 2009 (ML093310392), that there is no obvious mounding in the Lower Shallow Aquifer from the main cooling reservoir. Potentiometric surface maps of the Upper and Lower Shallow Aquifers are presented (1) in the COL FSAR, (2) in the applicant's groundwater model report dated November 30, 2009 (ML093360350), and (3) in the applicant's supplemental response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). Maps of the Shallow Aquifer potentiometric surfaces are also provided in the UFSAR for STP Units 1 and 2 (STPEGS, 2006).

Table 2.4S.12-1. Representative Properties of Confining Layers in the STP Hydrogeologic Strata (from FSAR Table 2.4S.12-14 and its proposed revision in the RAI response dated November 23, 2009)

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
Upper shallow aquifer confining layer	Thickness	m (ft)	6.1 (20) (pj)	3.1-9.1 (10–30)	Figure 2.4S.12-20
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08 (0.0005–0.05)	Table 2.4S.12-13
	Bulk (dry) density	kg/m^3 (pcf)	1,618 (101)	1,544-1,841 (96.4–114.9)	Table 2.4S.12-12
	Total porosity	%	40	31.8–42.8	Table 2.4S.12-12
Lower shallow aquifer confining layer	Thickness	m (ft)	6.1 (20) (pj)	4.6-7.6 (15–25)	Figure 2.4S.12-20
	Vertical hyd gradient	-	0.29	0.02–0.29	Table 2.4S.12-8
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08 (0.0005-0.05)	Table 2.4S.12-13
	Bulk (dry) density	kg/m^3 (pcf)	1,586 (99)	1,398-1,725 (87.3–107.7)	Table 2.4S.12-12
	Total porosity	%	42	36.1–47.2	Table 2.4S.12-12
Deep aquifer confining layer	Thickness	m (ft)	30.5 (100) (pj)	30.5-45.7 (100 –150)	Subsection 2.4S.12.3.1
	Vertical hyd cond	m/s (gpd/ft^2)	1.9E-09 (0.004) (gm)	2.4E-10-2.4E-08	Table 2.4S.12-13

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
				(0.0005-0.05)	
Deep aquifer confining layer (cont'd.)	Bulk (dry) density	kg/m^3 (pcf)	1,618 (101)	1,315-1,784 (82.1–111.4)	Table 2.4S.12-12
	Total porosity	%	41	33.4–51.8	Table 2.4S.12-12

*Values are arithmetic mean except where noted.
gm = geometric mean; am = arithmetic mean; pj = professional judgment; hyd = hydraulic; hyd cond = hydraulic conductivity; pcf = pounds per cubic foot.

Table 2.4S.12-2. Representative Hydrogeologic Properties of Aquifers in the STP Hydrogeologic Strata (from FSAR Table 2.4S.12-14 and its proposed revision in the RAI response dated November 23, 2009)

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
Upper Shallow Aquifer; Piezometric Surface 5 to 10 ft BGS	Thickness	m (ft)	7.6 (25) (pj)	6.1-9.1 (20–30)	Figure 2.4S.12-20
	Transmissivity	m^2/s (gpd/ft)	5.7E-03 (3,708) (gm)	1.7E-03-1.9E-02 (1,100–12,500)	Table 2.4S.12-10
	Storage coefficient	-	1.2E-03	1.7E-03 – 7E-04	Table 2.4S.12-10
	Horizontal hyd cond	m/s (gpd/ft^2)	7.8E-05 (165) (gm)	3.1E-05-2.0E-4 (65–420)	Table 2.4S.12-10
	Horizontal hyd gradient	-	0.002 (southeast) 0.0008 (southwest)	0.0007–0.002; 0.0005–0.0008	Subsection 2.4S.12.2.2
	Bulk (dry) density	kg/m^3 (pcf)	1,586 (99)	1,557-1,605 (97.2–100.2)	Table 2.4S.12-12
	Total porosity	%	41	39.5–41.7	Table 2.4S.12-12
	Effective porosity	%	33	31.6–33.4	Table 2.4S.12-12
Lower Shallow Aquifer; Piezometric Surface 10 to 15 ft BGS	Thickness	m (ft)	12.2 (40) (pj)	7.6-15.2 (25–50)	Figure 2.4S.12-20
	Transmissivity	m^2/s (gpd/ft)	2.8E-02 (18,209) (gm)	2.0E-02-5.1E-02 (13,000–33,150)	Table 2.4S.12-10
	Storage coefficient	-	5.8E-4	4.5E-4–7.1E-4	Table 2.4S.12-10
Lower Shallow Aquifer; Piezometric	Horizontal hyd cond	m/s (gpd/ft^2)	2.6E-04 (543) (gm)	1.9E-04-3.1E-04 (410-651)	Table 2.4S.12-10

Hydrogeologic Unit	Property	Units	Representative Value*	Range	FSAR Source
Surface 10 to 15 ft BGS (cont'd)	Hydraulic gradient	-	0.0007 (southeast)	0.0004–0.0007	Subsection 2.4S.12.2.2;
	Bulk (dry) density	kg/m^3 (pcf)	1,634 (102)	1,514-1,922 (94.5–120.0)	Table 2.4S.12-12
	Total porosity	%	39	28.8–43.9	Table 2.4S.12-12
	Effective porosity	%	31	23.0–35.1	Table 2.4S.12-12
Deep Aquifer	Thickness	m (ft)	243.8-304.8 (800-1000) (pj)		Ryder (1996), LCRA (2007b)
	Transmissivity	m^2/s (gpd/ft)	4.9E-02 (31,379) (gm)	3.7E-02-7.7E-02 (24,201–50,000)	Table 2.4S.12-10
	Storage coefficient	-	4.9E-4	2.2E-4–7.6E-4	Table 2.4S.12-10
	Horizontal hyd cond	m/s (gpd/ft^2)	2.0E-04 (420) (gm)	4.9E-05-1.9E-03 (103–3,950)	Table 2.4S.12-9
	Hydraulic gradient	-	0.002	0.0008–0.002	Subsection 2.4S.12.2.2
	Bulk (dry) density	kg/m^3 (pcf)	1,634 (102)	1,514-1,922 (94.5–120.0)	Lower Shallow Aquifer
	Total porosity	%	39	28.8–43.9	Lower Shallow Aquifer
	Effective porosity	%	31	23.0–35.1	Lower Shallow Aquifer

*Value = arithmetic mean except where noted.
gm = geometric mean; am = arithmetic mean; pj = professional judgment; hyd = hydraulic; hyd cond = hydraulic conductivity; pcf = pounds per cubic foot.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.1.4 and the proposed revision in the response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The staff confirmed that the applicant has addressed the relevant information. The staff's review of the application included documents on the hydrology and aquifers of the site; the Units 1 and 2 UFSAR (STPEGS, 2006); the staff's final environmental statements related to the operation of STP Units 1 and 2 (NRC 1975, 1986); and the applicant's responses to the RAIs cited above.

The applicant completed documenting additional characteristics of the site as a result of RAIs and commitments made during the acceptance review of the application. The staff's review found that the main cooling reservoir does influence the Upper Shallow Aquifer. The staff also noted that the backfilled excavation at STP Units 1 and 2 does influence the Upper and Lower Shallow Aquifers, and the pathways from proposed STP Units 3 and 4 will need to account for a similar influence at the backfilled excavation of the proposed units. A review of pre-site and site-startup conditions in the Lower Shallow Aquifer, as exhibited in the Units 1 and 2 UFSAR Revision 13, Figures 2.4.13-17 and 2.4.13-17a (STPEGS, 2006) compared to current piezometric levels and contours (see FSAR Figure 2.4S.12-19) and the applicant's proposed changes in the RAI response dated November 23, 2009 (ML093310392), led the staff to raise the issue that there are site influences on the Lower Shallow Aquifer. This issue is addressed in detail in Section 2.4S.12.4.7 of this SER.

The staff reviewed the FSAR and its proposed revisions in response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The staff found the applicant's description of site-specific hydrogeology acceptable for the following reasons: (1) the description of the proposed site for STP Units 3 and 4 is consistent with the description of the hydrology and aquifers underlying existing STP Units 1 and 2, and (2) the site characterization provides additional information on the aquifers underlying the proposed site of STP Units 3 and 4. The FSAR changes in response to RAI 02.04.12-28 were tracked as Confirmatory Item 02.04.12-1 in the SER with open items. The staff verified that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-28 and Confirmatory Item 02.04.12-1 are closed.

2.4S.12.4.3 Groundwater Sources and Sinks

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.5, the applicant describes the recharge and discharge areas of the regional groundwater system. Natural regional groundwater flow in the Beaumont Formation (i.e., including the Shallow Aquifer and Deep Aquifer) is from recharge areas northwest of the site toward the Gulf of Mexico or Colorado River alluvium. The main cooling reservoir also recharges the Upper Shallow Aquifer, as demonstrated by potentiometric levels that decrease in piezometers farther from the embankment. A series of 770 relief wells that penetrate the Upper Shallow Aquifer at the toe of the embankment was installed to capture at least 50 percent of the seepage from the main cooling reservoir, as stated in RAI 02.04.12-20 dated December 30, 2008 (ML083660390). Based on site characterization data, the applicant believes that the main cooling reservoir affects the groundwater flow direction in the Upper Shallow Aquifer, but the applicant does not detect any obvious mounding in the Lower Shallow Aquifer from the main cooling reservoir as stated in RAI 02.04.12-28 dated September 21, 2009.

The applicant describes the main cooling reservoir recharge to the Upper Shallow Aquifer as occurring mainly as seepage through the reservoir bottom. Design features of the main cooling reservoir embankment include a compacted low-permeability clay core, sand drainage blankets, and a series of 770 relief wells completed in the Upper Shallow Aquifer. Groundwater flow through the embankment and in the underlying aquifer is intercepted, in part, by the system of relief wells. The system of relief wells is designed (1) to ensure the stability of the embankment, and (2) to maintain potentiometric levels in the STP Units 1 and 2 power block below the ground surface. During the design of the main cooling reservoir, estimates of total seepage losses and intercepted groundwater were 7.031 M m^3/yr (5,700 ac-ft/yr) and 4.75 M m^3/yr (3,850 ac-ft/yr) (i.e., 68 percent intercepted), respectively.

Concentrated pumping from aquifers can alter or locally reverse the regional flow pattern. In the vicinity of the proposed facility, the production wells for existing plants have caused the Deep Aquifer to exhibit a local reversal of the flow pattern. This results in a radial flow toward the production wells from the surrounding aquifer.

In the vicinity of the site, the Holocene age alluvium is recharged by precipitation and by discharge from the Shallow Aquifer. Flow paths in the alluvium are generally short, because flow is intercepted by streams and rivers that incise the alluvial material.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.1.5 and confirmed that the applicant has addressed relevant information. In the review of the application, the staff also reviewed documents about the hydrology and aquifers of the region from the USGS (Ryder, 1996; Ryder and Ardis, 2002); the TWDB (Hammond, 1969; Mace et al., 2006); and the LCRA (Young et al., 2007); in addition to documents submitted by the applicant about the site hydrology and environment (STPEGS, 2006; Reynolds, 2007; Sherwood and Travis, 2007, 2008) and documents from the NRC about the site-specific hydrology (NRC, 1975; NRC, 1986). The staff concluded that the applicant's description of groundwater sources and sinks is consistent with this body of work.

The staff noted the USGS (Ryder, 1996) observations that Matagorda County is in a region of several counties where the greatest amount of groundwater pumping is relatively near the outcrop where the aquifer is recharged and therefore, recharging provides a source to balance the large groundwater withdrawals. This balance was of special interest because of the irrigation of rice in the vicinity of both the pumping and the recharging. Ryder (1996) noted that in areas of little or no pumping, essentially in areas where pre-development conditions persist, the recharge rate is generally between 0 and 2.54 cm/yr (0 and 1 in./yr). During periods of drought, Young et al., (2007) described the average recharge rate as 3.56 to 4.32 cm/yr (1.4 to 1.7 in./yr) and during a wet year, the recharge rate is 11.68 cm/yr (4.6 in./yr). Ryder (1996) also stated that recharge rates increase between 10.2 and 15.2 cm/yr (4 and 6 in./yr) in the rice irrigation areas. As a result, Ryder (1996) concluded that the drawdown was not large in the region (less than 15.2 m [50 ft]) because withdrawals by pumping were balanced by an increase in recharge rates over pre-development levels.

The staff found the applicant's description of the groundwater sources and sinks acceptable because the sources and sinks identified for the Upper and Lower Shallow Aquifer and the Deep Aquifer are consistent with those identified by the USGS, the TWDB, the LCRA, and site-specific documents.

2.4S.12.4.4 Plant Groundwater Use

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.1.6, and its proposed revisions in the response to RAI 02.04.12-28 dated September 21, 2009 (ML092710096), the applicant describes the operation of STP Units 1 and 2 as currently using groundwater from five production wells. Annual groundwater usage at STP Units 1 and 2 from 2001 through 2006 was 1.59 M m^3/yr (1,288 ac-ft/yr) (ML092710096). Groundwater use for STP Units 1 and 2 includes supplies for the makeup of the demineralized water system, the potable and sanitary water system, and the fire protection system. Groundwater use for the proposed STP Units 3 and 4 includes similar plant operation water supplies and makeup water to the UHS. The applicant projects (ML092710096) that the normal groundwater consumption rate for the proposed units is 1.94 M m^3/yr (1,575 ac-ft/yr), and the maximum short-term groundwater demand is expected to be as great as 6.83 M m^3/yr (3,434 gpm or 5,547 ac-ft/yr). The groundwater supply wells associated with the proposed STP

Units 3 and 4 will not be a safety-related water source because the UHS has a 30-day supply of water, which is sufficient for a plant shutdown without an additional water supply. After studying the plant water use and the site groundwater use issues, the applicant found that the current groundwater use permit limit of 11.1 M m^3 (9,000 ac-ft) during the approximate 3-year permit period is adequate for the operation of STP Units 1 and 2 and the construction, testing, startup, and operation of STP Units 3 and 4.

In FSAR Subsection 2.4S.12.3.3 and its proposed revisions (ML092710096), the applicant describes the proposed groundwater use in light of the existing groundwater permit and groundwater use by the existing STP Units 1 and 2. During the construction of the proposed plant, groundwater will be used for the potable and sanitary water supply, the manufacture and placement of concrete, dust control, backfill moisture, and testing and flushing. During plant operation, groundwater will be used for the potable and sanitary supply, the production of demineralized water, fire protection, and makeup water for the UHS. The groundwater use permit held by the applicant is for 11.1 M m^3 (9,000 ac-ft) during the period of the permit, which is approximately 3 years. For discussion purposes, this use amounts to approximately 3.7 M m^3/yr (3,000 ac-ft/yr) or a normalized continuous pumping rate of 7,040 liters per minute (Lpm) (860 gpm). The relevant sections of the ER in the COL application Part 3, describe details of onsite plant groundwater use and the effects. However, these groundwater uses, including makeup water for the UHS, are not safety-related functions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.1.6 and its proposed revisions. In June 2009, the staff's review of FSAR confirmed that the applicant has addressed the relevant information topics. However, the information in the STP FSAR is not consistent with that found in related sections of the STP ER. RAIs related to the ER (RAI 05.10-04) and the FSAR (RAI 02.04.12-36) were issued to resolve the inconsistencies.

The applicant's responses to ER RAI 05.10-04 dated September 28, 2009 (ML092730285), and to FSAR RAI 02.04.12-36 dated September 21, 2009 (ML092710096), provide groundwater use rates for STP Units 1 and 2 under normal and outage conditions, and for STP Units 3 and 4 under normal and maximum conditions. Furthermore, the applicant stated that the existing groundwater permit limit provides an adequate water supply for the operation of STP Units 1 and 2 and the construction, initial testing, and operation of STP Units 3 and 4. The applicant stated that the water-storage capacity will be provided to supply the groundwater for peak site water demands, and the main cooling reservoir and the Colorado River remain as alternative sources to meet unanticipated peak site water demands.

After reviewing the applicant's responses above and the calculation package on future STP groundwater use, the staff concluded that the applicant's description of plant groundwater use is accurate. The staff noted that STP groundwater wells are not a safety-related source of water for STP Units 3 and 4.

The staff reviewed FSAR Subsection 2.4S.12.3.3 and its proposed revisions dated September 21, 2009 (ML092710096). Based on the applicant's analysis of the groundwater requirements during the construction and operation of the proposed plant, it is apparent that the operation of STP Units 1 and 2 and the construction, testing, and operation of STP Units 3 and 4 can be accomplished using the applicant's currently held groundwater use permit. If additional water is needed to meet maximum short-term groundwater demands for the operation of STP Units 1 and 2 and the construction, testing, and operation of the proposed STP Units 3 and 4, then the main cooling reservoir and Colorado River water are available under the applicant's existing contracts. The applicant states that one or more new groundwater production wells will be constructed to decrease pumping rates at wells, distribute drawdown affects, and ensure a sufficient withdrawal capacity to serve the total site groundwater demand under the existing groundwater permit (ML092710096 and ML092730285). Although specific locations of the new wells have not been provided, the applicant has provided the required separation distances from the existing and proposed reactors and from offsite wells. Groundwater supplies for the proposed STP Units 3 and 4 are not safety related.

The staff concluded that the applicant's description of plant groundwater use and effects is a consistent and acceptable representation of its intended groundwater use. The FSAR Subsection 2.4S.12.3.3 changes in response to RAI 02.04.12-36 (ML092710096) were tracked as Confirmatory Item 02.04.12-1 in the SER with open items. The staff verified that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-36 and Confirmatory Item 02.04.12-1 are closed.

2.4S.12.4.5 Historical and Projected Groundwater Use

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.1, the applicant describes the historical and projected groundwater uses in Matagorda County. In Section 2.3 of the COL application ER, the applicant also provides details of historical and projected groundwater uses. Table 2.4.12-3 summarizes the quantity of groundwater permitted and the various estimates of the groundwater resource in Matagorda County. The annual quantity of groundwater permitted by the Coastal Plains Groundwater Conservation District (CPGCD) exceeds the current estimates of managed available groundwater and the estimated groundwater supply. The permitted use also exceeds recorded usage within the county. The CPGCD notes that little science has been applied to estimating the managed available groundwater resource adopted in the site's groundwater management plan (Turner, Collie, and Braden, Inc., 2004), and caution should be exercised in using this value (i.e., 115,528 Lpm [30,520 gpm or 49,221 ac-ft/yr]). It is apparent that satisfying the current annual permitted amount within the CPGCD would require investment in infrastructure, including the construction of wells and delivery systems. Satisfying the future demand level in 2060 would also require investment and could be based on water-conservation strategies and desalination of either sea water or brackish groundwater.

The infrastructure is in place at the STP site to fully use its permit limit, and although it has not been fully used to date, it is included in the estimated groundwater supply value of 83,992 Lpm ([22,189 gpm or 35,785 ac-ft/yr]). The full STP permit limit is included in the annual permitted value.

Table 2.4S.12-3. Groundwater Resource Estimates for Matagorda County

Resource Description	L/s (gpm)	m^3/yr (ac-ft/yr)	Reference*
Managed available groundwater	1,925 (30,520)	6.1E+07 (49,221)	TC&B 2004, Table 1
Estimated groundwater supply	1,400 (22,189)	4.4E+07 (35,785)	TC&B 2004, Table 4
Average groundwater use 1980–2000	1,183 (18,746)	3.7E+07 (30,233)	TC&B 2004, Table 2
High groundwater use–1988	1,707 (27,055)	5.4E+07 (43,634)	TC&B 2004, Table 2
Low groundwater use–1998	554 (8,783)	1.8E+07 (14,165)	TC&B 2004, Table 2
Future demand–Total in 2060	2,556 (40,509)	8.1E+07 (65,331)	LCRWPG 2006
Annual permitted (2008–2010)	2,006 (31,800)	6.3E+07 (51,285)	CPGCD 2009
*TC&B = Turner, Collie, and Braden, Inc.; LCRWPG = Lower Colorado River Water Planning Group; CPGCD = Coastal Plains Groundwater Conservation District			

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.1 and the applicant's response to RAI 02.04.12-36 dated September 21, 2009 (ML092710096). The staff's review confirmed that the applicant has addressed the relevant information. In the review of the application, the staff also reviewed documents on Texas water law (Texas Water Code Chapter 36, Groundwater Conservation Districts) and water management documents at the local, regional, and State levels (Turner, Collie, and Braden, Inc., 2004; LCRWPG, 2006; LCRWPG, 2009; TWDB, 2006).

After the site audit in 2008, the applicant was asked to revisit the topic of historical and projected groundwater uses. Information in the application was thought to be dated and as such, it might not have reflected the current groundwater use and availability in Matagorda County. The applicant's response to RAI 02.04.12-04 dated July 2, 2008 (ML081890239), which requested groundwater use projections for the region that are consistent with the license period, was reviewed and accepted by the staff. The values cited in this response for available groundwater, 60.71 M m^3/yr (49,221 ac-ft/yr); average groundwater use between 1980 and 2000, 37.29 M m^3/yr (30,233 ac-ft/yr); and available groundwater supply, 44.14 M m^3/yr (35,785 ac-ft/yr) are from the agency responsible for assessing the groundwater resources in Matagorda County and for issuing groundwater use permits (i.e., the Coastal Plains Groundwater Conservation District [Turner, Collie, and Braden, Inc., 2004]). At this time, these values are current and as issued by the authorized body.

The staff concluded that the applicant's description of historical and projected groundwater use is an accurate representation of groundwater use in the vicinity of the STP site.

2.4S.12.4.6 Ground water Flow Directions

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.2 and its proposed revisions (ML093310392), the applicant describes how the regional deep aquifer flow directions vary over time because of changes in regional and local groundwater withdrawal patterns. In 1967, the groundwater heads were 6.1 to 9.1 m (20 to 30 ft) above MSL in the northern portion of Matagorda County and sloping to sea level at Matagorda Bay. Localized flow disturbances were evident at that time within Matagorda County, which caused an elevated head of more than 12.2 m (40 ft) above MSL and depressions of -33.53 m (-110 ft) below MSL.

While regional potentiometric-level maps are not available for the Shallow Aquifer, local data sets are available from the existing STP site piezometers for STP Units 1 and 2 and the site characterization effort completed for STP Units 3 and 4 (see FSAR Figures 2.4S.12-19 and its proposed revision in ML093310392) showing quarterly data from February 2007 through December 2008). The flow direction in the Upper Shallow Aquifer is described as having components to the east and southeast toward the Colorado River and to the south and southwest along the west side of the main cooling reservoir (ML093310392). In the Lower Shallow Aquifer, the flow direction is described as predominantly toward the east and southeast. The applicant has interpreted the data since September 2008 to indicate that there is no obvious mounding from the main cooling reservoir observed in the Lower Shallow Aquifer (ML092710096).

The recent data indicate that at certain times of the year and at points in the vicinity, there is an upward gradient to Kelly Lake from the Upper Shallow Aquifer, and an upward gradient from the Lower to the Upper Shallow Aquifer is possible (ML083660390). However, at other times of the year and at points in the vicinity of Kelly Lake, the gradients are downward. Thus, there appears to be a seasonal variation (ML102450252), and it is not clear that Kelly Lake is a groundwater discharge location (ML092710096). However, for groundwater flow directions to the east and southeast, the applicant included exposure points at the site boundary, at a private well (i.e., well 2004120846), and at the Colorado River. Although points downgradient of the site boundary to the southeast—including the unnamed tributary feeding Kelly Lake, a private well, Kelly Lake, and the Colorado River—are all plausible, they are conservatively represented by a hypothetical well at the site boundary (ML092710096).

Representative values and ranges of groundwater gradients are taken from the preconstruction potentiometric surfaces for the flow directions considered by the applicant and included in Table 2.4S.12-2 of this SER.

Post-construction groundwater simulations show a groundwater depression in the vicinity of the power block in the Upper Shallow Aquifer, with releases into that aquifer moving downward into the Lower Shallow Aquifer before migrating to the site boundary (ML102450252 and ML103540324). Field observations at STP Units 1 and 2 of tritium in groundwater and the potentiometric surface confirm this behavior (ML092710096 and ML102450252). Releases into the Lower Shallow Aquifer are projected to move to the east-southeast and to cross the eastern site boundary (ML103540324).

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.2 and its proposed revisions (ML093310392 and ML092710096) and confirmed that the applicant has addressed the relevant information topics.

During a June 2009 review of FSAR and the RAI responses, it was the view of the staff that groundwater flow away from the proposed Units 3 and 4 was plausible to both the southeast and southwest in the Shallow Aquifer. This view was based on site characteristics documented by the applicant that showed groundwater mounding in the Upper Shallow Aquifer and an absence of similar data on the Lower Shallow Aquifer. And in the future, the higher hydraulic head of the Upper Shallow Aquifer will be in direct communication with the Lower Shallow Aquifer, because the excavation within the powerblock will remove the confining strata separating them. The staff received the responses to RAIs (ML092710096, and ML093310392), which included amendments to the FSAR. The staff reviewed the FSAR, the applicant's proposed revisions in these responses, and the revised groundwater model document and found the main cooling reservoir influence and pre-construction (i.e., pre-STP Units 3 and 4) groundwater flow directions are well characterized by the applicant. However, the staff believed the post-construction setting required further evaluation before all plausible future groundwater flow directions could be identified or discarded. Accordingly, in April 2010, the NRC issued supplemental RAIs. The applicant submitted an additional analysis of the post-construction setting (ML102450252). The staff reviewed the RAI responses and noted that the post-construction setting may be well described by three plausible pathways. The applicant provided field data and simulations justifying the exclusion of a west-southwest pathway in the Lower Shallow Aquifer. The FSAR and its revisions include four pathways: (1) the Upper Shallow Aquifer to the east-southeast site boundary, (2) the Upper and Lower Shallow Aquifer to the east-southeast and an existing well, (3) both Shallow Aquifer units to the Colorado River, and (4) a potential Upper Shallow Aquifer discharge to the west-southwest and Little Robin Slough. The staff tracked the applicant's additional sensitivity cases that address other aspects of the Lower Shallow Aquifer pathway to the west-southwest as Open Item 2.4.12-1 in the SER with open items.

Additional sensitivity cases that addressed aspects of the Lower Shallow Aquifer pathway to the west-southwest were submitted by the applicant in a letter dated December 15, 2010 (ML103540324). The staff reviewed the supplemental RAI response (ML103540324) and the groundwater documentation (ML110140173). The staff concluded that the Lower Shallow Aquifer flows from the proposed units to the east-southeast site boundary. Based on the site characterization and pre- and post-construction model simulations of the Shallow Aquifer, the staff accepted the applicant's groundwater flow direction. This closes the groundwater flow direction aspect of Open Item 2.4.12-1 in Subsection 2.4S.12.4.12 of this SER.

2.4S.12.4.7 Temporal Groundwater Trends

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.3, the applicant presents long-term records for two Deep Aquifer wells monitored by the TWDB that reveal (1) an upper Deep Aquifer that is recovering to 1957 *levels (18.29 m [60 ft] BGS); and (2) a lower Deep Aquifer, which is the screened interval of* the STP wells that is exhibiting a steady groundwater level (6.1 to 9.1m [20 to 30 ft] BGS).

The applicant presents water levels within the Upper and Lower Shallow Aquifer for the period from 1994 through 2006. They reveal a high groundwater level in the Upper Shallow Aquifer of approximately 8.23 m (27 ft) MSL adjacent to the site boundary to the east and west of proposed Units 3 and 4. Since early 1997, the variation in the groundwater level of the Upper Shallow Aquifer has been approximately 1.83 m (6 ft). Observation well 929U, completed in the Upper Shallow Aquifer to the northeast of proposed Unit 3, shows a peak groundwater elevation of 8.38 m (27.49) ft MSL. Observation well 993U, between proposed Unit 3 and the main cooling reservoir, shows a peak groundwater elevation of 7.928 m (26.01 ft) MSL. Well 602A, completed in the Lower Shallow Aquifer and immediately north of the proposed units, shows a peak groundwater elevation of 7.86 m (25.8 ft) MSL and a variation of approximately 1.22 m (4 ft). Data collected during the STP Units 3 and 4 site characterization efforts reveal between 0.61 and 1.95 m (2.8 and 6.4 ft) of variation from December 2006 through September 2008 in the Upper Shallow Aquifer and between 0.61 and 1.22 m (2.6 and 4.0 ft) of variation in the Lower Shallow Aquifer. Groundwater-level data for the Shallow Aquifer show that levels in the Upper Shallow Aquifer are consistently higher than those in the Lower Shallow Aquifer and within approximately 1.5 m (5 ft) BGS during the site characterization period.

During 2007, the Upper Shallow Aquifer piezometric level was steady with a slight decrease after August. The Lower Shallow Aquifer exhibited an increase in the piezometric level until August and then a decrease through December. During 2008, a steadily decreasing trend in piezometric levels was seen in both Shallow Aquifers. This reflected drought conditions in the region.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.3 and its proposed revisions (ML092710096 and ML093310392) and confirmed that the applicant has addressed the relevant information topics.

The applicant's response to RAI 02.04.12-28 dated August 5, 2009 (ML092170354), which requested the applicant to update the FSAR sections affected by updated data sets with regard to plausible pathways, mounding, gradients, and maps, was completed in the supplemental response dated September 21, 2009 (ML092710096). This RAI response relied, in part, on groundwater model simulations that were revised and submitted later by the applicant. During the review of the FSAR, its proposed revisions, and the groundwater model documentation, the staff concluded that the influence on future mounding in the Lower Shallow Aquifer may have been masked by model bias, and a future hydraulic gradient to the west or southwest may not have been identified. Therefore, a potentially important change in the groundwater system resulting from building the proposed plant may have been omitted. Accordingly, the staff issued RAIs requesting additional information. In a letter dated August 30, 2010, the applicant provided responses to these RAIs (ML102450252).

The staff's review of the applicant's responses clarified the potential for mounding in the Lower Shallow Aquifer, and the potential for a west-southwest directed pathway in the Lower Shallow Aquifer during the post-construction period. The staff identified that field observations of the potentiometric surface in the Upper and Lower Shallow Aquifers in the vicinity of the STP Units 1 and 2 excavation and fill show that removal of the confining sediments between these two aquifers resulted in a groundwater depression in the Upper Shallow Aquifer and a slight groundwater mound in the Lower Shallow Aquifer. These changes occur in the immediate vicinity of the excavation at Units 1 and 2. The staff concludes that a similar response to excavation and fill at the proposed location of Units 3 and 4 can be anticipated. Post-construction simulations of Units 3 and 4 exhibit this behavior; however, the staff found that a west-southwest pathway is not projected to occur in the Lower Shallow Aquifer. Additional sensitivity cases further demonstrated the response to excavation and fill, and the absence of a west-southwest pathway post-construction (ML103540324). This closes the temporal groundwater trends aspect of Open Item 2.4.12-1 described in Subsection 2.4S.12.4.12 of this SER.

The staff reviewed FSAR Subsection 2.4S.12.2.3 and its proposed revisions, and the final sensitivity cases that address aspects of the Lower Shallow Aquifer pathway to the west-southwest (ML103540324) and concluded that the applicant has accurately described the groundwater trends that can be expected at the STP site. Of the trends identified, the staff concur with the normal trend of groundwater rise and decline in response to seasonal change; the trend of declining piezometric levels in response to drought conditions; the anticipated change in the groundwater piezometric levels in response to removing the confining zone materials that separate the Upper and Lower Shallow Aquifers; and the effect of the main cooling reservoir on the Upper Shallow Aquifer.

2.4S.12.4.8 Aquifer Properties

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.4, the applicant presented information about precipitation, transmissivity, storativity, hydraulic conductivity, porosity, effective porosity, and bulk density. The applicant also presented the average annual precipitation as 106.7 cm (42 in.) based on a 30-year record from 1951 through 1980 in the vicinity of the STP site. Annual runoff was estimated at approximately 30 cm (12 in.), with the remaining 76.2 cm (30 in.) attributed to the combination of evaporation, transpiration, and infiltration that becomes the recharge to the underlying aquifer. Much of the 76.2 cm (30 in.) is recycled to the atmosphere through evapotranspiration.

The applicant divided the properties of the aquifer between hydrogeological and geotechnical parameters. Transmissivity, storage coefficient, and hydraulic conductivity from tests conducted in the field or derived from such tests are among the hydrogeological parameters. Bulk density (or dry unit weight), porosity, effective porosity, and permeability from grain-size distributions estimated from laboratory tests are presented as geotechnical parameters.

In FSAR Subsection 2.4S.12.2.4.1, the applicant provides a review of the site-specific measurements and data reductions for hydrogeological parameters against regional parameters shown by Hammond (1969). The applicant concludes that site measurements of Deep Aquifer transmissivity fall within the range of regional values. However, those for the Shallow Aquifer fall below the regional range as a result of a pair of low field measurements in the Upper Shallow Aquifer. The applicant finds that all storage coefficient values fall within the regional range. The applicant compares hydraulic conductivities for the Shallow Aquifer derived from transmissivity measurements and inferred aquifer thickness and slug tests. Hydraulic conductivities for the Shallow Aquifer derived from slug test data were found to fall somewhat below the regional range; however, geometric means of hydraulic conductivity from the two approaches were comparable. Final compilation of the transmissivity and hydraulic conductivity data by the applicant relied on both the FSAR and its proposed revisions (ML09331092). Technical justification for the aquifer pumping test results included in and excluded from the compilation of hydraulic conductivity, transmissivity, and storativity data were provided by the applicant in the supplemental response to RAI 02.04.12-38 (ML102450252).

In FSAR Subsection 2.4S.12.2.4.2, the applicant describes how geotechnical parameters were determined directly or indirectly from laboratory data and reported bulk density as measured in the laboratory. Porosity was calculated using a conversion from void ratio, and effective porosity was estimated as a specific yield using a graphical method relating median grain size to specific yield. Permeability was estimated from grain size using the Hazen approximation, and the applicant found that the geometric mean of hydraulic conductivity values was similar to but lower than that for the STP slug test results. The applicant reports the hydraulic conductivity of clay strata measured during the site characterization effort conducted for STP Units 1 and 2.

The saturated hydraulic conductivity derived from aquifer pumping tests yielded higher geometric means than those derived from slug tests. Therefore, it is the aquifer pumping test results that appear in the applicant's summary table reporting representative properties of the hydrogeologic units (see FSAR Table 2.4S.12-14). Tables 2.4S.12-1 and 2.4S.12-2 of this SER contain a summary of representative property values and their ranges for all geohydrologic strata.

<u>NRC Staff's Technical Evaluation</u>

The staff reviewed FSAR Subsection 2.4S.12.2.4, its proposed revisions (ML093310392), and groundwater model documentation. The staff's review confirmed that the applicant has addressed relevant information. The staff also reviewed documents about the hydrology and aquifers of the region from the USGS (Ryder, 1996; Ryder and Ardis, 2002), the TWDB (Hammond, 1969; Mace et al., 2006), and the LCRA (Young et al., 2007).

Most of the documents reviewed by the staff report on the Gulf Coast or Coastal Lowlands Aquifer system over a larger region and throughout its depth, especially at inland locations where fresh water has been pumped from deeper strata within the system than are available in Matagorda County. The analysis at the STP site is more local and relies on measurements made locally. However, the staff concluded that data sets of the documented regional models support the aquifer properties found at the STP site, which are summarized in FSAR Table 2.4S.12-14 and shown in Tables 2.4S.12-1 and 2.4S.12-2 of this SER.

The applicant's information about aquifer properties in the FSAR resulted in five RAIs requesting consistent interpretations of the data. Responses to these RAIs gave rise to additional RAIs including RAI 02.04.12-28, which requested the applicant to incorporate new information into the application. The applicant issued revisions to the FSAR (ML093310392) and a revised groundwater model. The staff's review of the FSAR, its proposed revision (FSAR Revision 3), and the revised groundwater model identified inconsistencies in the description of site-specific hydraulic conductivity and transmissivity data. The staff issued RAI 02.04.12-38 (ML101060021) requesting the applicant to correct these deficiencies, and the staff reviewed the applicant's responses (ML101390277 and ML102450252). Based on the applicant's technical justification of the aquifer pumping tests included in and excluded from the data compilation (e.g., hydraulic conductivity, transmissivity, storativity, porosity, and bulk density), and the staff's review of these data compared to other studies conducted on the aquifer system, the staff accepted the aquifer properties as representative of the Shallow and Deep Aquifer systems underlying the STP site.

2.4S.12.4.9 Hydrogeochemical Characteristics

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.2.5, the applicant provides groundwater quality data for the Deep Aquifer from the mid-1960s, mid-1970s, early 1980s, and one sample from 1991. The applicant also provides data for the Shallow Aquifer from the early 1970s and December 2006. The water quality data are consistent over time and suggest a groundwater system that is not experiencing substantial change. Both aquifers exceed the Secondary Drinking Water Standards (EPA, 2009b) for TDS and chloride. However, there are higher concentrations in the Shallow Aquifer.

The groundwater quality of each aquifer provides a signature that can be used to identify hydraulic connections between aquifers. Within this aquifer system, the Upper Shallow Aquifer is a sodium chloride type while both the Lower Shallow and Deep Aquifers are the sodium bicarbonate type. The Lower Shallow Aquifer exhibits a sodium chloride groundwater type at two onsite locations between the proposed reactors and the Colorado River. This suggests a localized hydraulic connection allowing groundwater from the Upper Shallow Aquifer to enter the Lower Shallow Aquifer. This could be a result of natural (e.g., discontinuous confining unit, incised channel, or scour) or manmade (e.g., pervious backfill, or leaking well) features.

In FSAR Subsection 2.4S.12.2.2, the applicant notes that the hydrogeochemical characteristics of the Shallow Aquifer compared to those of the main cooling reservoir water suggest that there is no strong geochemical correlation between the main cooling reservoir water and the groundwater north of the main cooling reservoir (i.e., in the vicinity of the existing and proposed STP units). In addition, the potentiometric maps indicate little evidence of groundwater mounding north of the main cooling reservoir. This data suggests that relief wells are effective in reducing seepage from the main cooling reservoir to the surrounding groundwater.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.5, and confirmed that the applicant has addressed relevant information. The staff also reviewed documents containing a discussion of the chemical characteristics of aquifers in the region from the USGS (Ryder and Ardis, 2002) and the TWDB (Hammond, 1969; Mace et al., 2006). The staff also referred to the EPA Primary and Secondary Drinking Water Standards (EPA 2009a; EPA, 2009b). While evaluating and discussing the chemical characteristics on a larger scale, these documents support the applicant's evaluation of hydrogeochemical characteristics in the vicinity of the STP site.

Based on the information in the FSAR, the staff noted that the applicant's description of the hydrogeochemical characteristics of the groundwater resource is an accurate description of groundwater quality in the vicinity of the STP site. Therefore, the staff concluded that the interaction between the Upper and Lower Shallow Aquifers and the main cooling reservoir is a localized phenomenon in the vicinity of the STP site. Therefore, RAI 02.04.12-22 is closed.

2.4S.12.4.10 Subsurface Pathways

In FSAR Subsection 2.4S.12.3, and its proposed revisions (ML093310392), the applicant evaluates subsurface pathways to an offsite receptor. Information provided by the applicant includes an evaluation of alternative pathways, an assessment of advective travel times, and results from a model of post-construction groundwater flow conditions.

Alternative Pathways Evaluation

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.3.1, the applicant interprets field data to assert that the most plausible groundwater pathway for a release from STP Units 3 and 4 is to the east and the southeast in both the Upper and Lower Shallow Aquifer. The applicant also acknowledges a plausible flow component toward the southwest in the Upper Shallow Aquifer.

The excavation for STP Units 3 and 4 penetrates the confining zone separating the Upper and Lower Shallow Aquifer. The hydraulic gradient in the undisturbed system is downward from the Upper to the Lower Shallow Aquifer. Because postulated accidental releases into the Upper Shallow Aquifer in the vicinity of the power block excavation and fill would move downward into the Lower Shallow Aquifer, the applicant concluded that the most likely groundwater pathway is the Lower Shallow Aquifer.

Offsite migration pathways for the Upper Shallow Aquifer are to the southeast with the exposure point at (1) the eastern site boundary or an unnamed tributary flowing into Kelly Lake, (2) a private well, and (3) the Colorado River. Kelly Lake is also plausible. The applicant used an exposure point on the site boundary to the east of proposed Unit 3 in the Upper Shallow Aquifer to conservatively represent exposures. The fourth pathway, a southwest pathway in the Upper Shallow Aquifer, is noted to discharge into the headwaters of LRS or into a hypothetical domestic water well installed offsite. The applicant used an exposure point on the site boundary to the west of proposed Unit 4 in the Upper Shallow Aquifer. Offsite migration pathways and exposure points for the Lower Shallow Aquifer are to the east-southeast and are the same as those described for the Upper Shallow Aquifer. The nearest exposure point and, therefore, the most conservative one is a hypothetical domestic water well completed on the eastern site boundary.

The applicant views the Deep Aquifer as the least likely pathway because of the low permeability confining zone separating the Shallow and Deep Aquifers. Releases would likely move to exposure points in the Lower Shallow Aquifer instead of entering and moving through the confining zone. A release that would penetrate the confining zone and enter the Deep Aquifer would be drawn to the production wells, thereby minimizing the potential for offsite migration and exposure. The applicant concluded that there is no credible pathway for offsite exposure involving the Deep Aquifer.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.3.1, its proposed revisions, and revised groundwater model documentation. The staff's review confirmed that the applicant has addressed the relevant information topics.

The applicant's response to RAI 02.04.12-17 dated August 12, 2008 (ML082270381), which requested an evaluation of the potential for buoyancy and chelating agents to influence pathways and mobility, was reviewed by the staff and two supplemental RAIs were issued (RAIs 02.04.12-29 and 02.04.12-30) to further evaluate and clarify issues with respect to buoyancy and chelation. While providing a further analysis of the potential for buoyancy to influence pathways, the issue of buoyancy is removed by the inclusion of pathways in the Upper and Lower Shallow Aquifer to the east-southeast and in the Upper Shallow Aquifer to the west-southwest. The applicant adopted the receptor locations on the eastern and western boundaries of the site because they provide the shortest pathways. The applicant showed the receptor location on the eastern boundary of the site in the Upper Shallow Aquifer to be the pathway with the shortest travel time.

Regarding chelating agents, the staff found that the applicant's technical basis is sound for neglecting the potential influences of chelating agents, because it is based on published literature on chelator influences on sediment adsorption, on the expected use and disposal of chelating agents, and on site-specific considerations of substantial dilution by groundwater during any release and an abundance of competing cation clay and silt minerals. RAIs 02.04.12-17, 02.04.12-29, and 02.04.12-30 are considered closed.

The staff reviewed and accepted the applicant's response to RAI 02.04.12-22 dated July 16, 2008 (ML082030326), which requested clarification regarding justification for excluding the Deep Aquifer as a plausible pathway. The applicant states that downward migration from the release point into the Deep Aquifer is not plausible because (1) transport will occur in the media of least resistance (i.e., laterally in the Shallow Aquifer); (2) a 30- to 46-m (100- to 150-ft)-thick confining zone would need to be traversed over a prolonged period of time, and (3) a Deep Aquifer pump test demonstrated the hydraulic isolation of the Deep Aquifer from the Shallow Aquifer. The staff agreed with the applicant's justification to exclude the Deep Aquifer pathway. Therefore, RAI 02.04.12-22 is considered closed.

The staff reviewed and accepted the applicant's response to RAI 02.04.12-30 dated September 21, 2008 (ML092710096), which requested information about a release to the groundwater environment that could support or refute the conceptual model of downward migration of a liquid radioactive release within the Shallow Aquifer. Tritium concentration data from several Upper and Lower Shallow Aquifer well pairs support the conceptual model of downward migration in the vicinity of STP Units 1 and 2, in response to an unplanned release into the Upper Shallow Aquifer. The concept of a downward migration in the vicinity of the excavation and fill of STP Units 1 and 2 is further substantiated by the applicant's response to RAI 02.04.12-42 dated August 30, 2010 (ML102450252), which provided potentiometric data showing a groundwater depression in the Upper Shallow Aquifer and a groundwater mound in the Lower Shallow Aquifer in the vicinity of the existing units. The staff concurred that a downward migration between the Upper and Lower Shallow Aquifers is likely to occur at the proposed location of Units 3 and 4, because construction of the proposed units requires a similar excavation and fill. Therefore, RAI 02.04.12-42 is considered closed.

In early 2010, the staff concluded that additional efforts would be required before finalizing the exposure point and pathway evaluation. The staff reviewed the FSAR, its proposed revisions, and the groundwater model documentation. The rationale for the exclusion of a west-southwest pathway in the Lower Shallow Aquifer from proposed Unit 4 was not fully supported. It was apparent that the piezometric head in the Lower Shallow Aquifer could be higher after construction of the plant than measured in the pre-site characterization. The applicant noted in its supplemental response to RAI 02.04.12-28 (ML093310392) that during the site characterization, the west-southwest hydraulic gradient was small and was influenced by seasonal and climatic variability. However, the staff noted that the applicant's interpretation and rejection of a west-southwest pathway from proposed Unit 4 is not based on the possibility of a higher post-construction piezometric head. The staff also noted that the applicant's groundwater model could include a bias that acted to reduce estimates of a future hydraulic gradient to the west or southwest from proposed Unit 4. In addition, the staff noted that the applicant had not evaluated the potential for the permanent, low permeability Crane Foundation Retaining Walls (CFRWs) to influence the groundwater pathways and exposure points. In April 2010, the staff issued supplemental RAIs. Responses were received on August 30, 2010 (ML102450252) and supplemental responses to several RAIs were received on December 15, 2010 (ML103540324).

The applicant's responses to the April 2010, RAIs resulted in a revised and improved preconstruction groundwater model (i.e., new topographic data and revised general head boundary [GHB] conditions) and post-construction groundwater simulations based on several updates and design information on structures and structural fill, powerblock finished grade and backfill cover, slurry wall designs, the design of two CFRWs, the relocated MDC, and a conservative representation of the main cooling reservoir water height. Consistent with the tritium and piezometric head observations at STP Units 1 and 2, the post-construction model projects at the proposed Units 3 and 4 site a groundwater depression in the Upper Shallow Aquifer and a groundwater mound in the Lower Shallow Aquifer. Despite projecting a groundwater mound in the Lower Shallow Aquifer (ML102450252), the simulations did not project a west-southwest pathway from proposed Unit 4. The applicant noted that the model exhibits a bias in the piezometric head toward predicting a southwest pathway (ML102450252). The applicant's supplemental responses (ML103540324) provide an alternative groundwater conceptual model and several sensitivity cases. Based on review of the supplemental responses and groundwater model documentation (ML110140173), the staff accepted the responses and concluded that a west-southwest pathway in the Lower Shallow Aquifer is not plausible. Also, the staff concluded that the most plausible future groundwater pathway from the proposed units is in the Lower Shallow Aquifer toward the eastern site boundary.

Therefore, staff found the applicant's description of the alternative groundwater pathways acceptable based on the site characterization of the geohydrology of the site and the preconstruction and post-construction groundwater model simulations that identify the alternative pathways. This review closes the alternative pathways evaluation aspect of Open Item 2.4.12-1 in Subsection 2.4S.12.4.12 of this SER.

Advective Travel Times

<u>*Information Submitted by Applicant*</u>

In FSAR Subsection 2.4S.12.3.2, and its proposed revisions, the applicant provides the analysis of the travel time of groundwater along plausible alternative pathways. The applicant's analysis assumes a one-dimensional advective transport of groundwater and associated radioactive contaminants. Such an analysis of contaminant movement assumes that the contaminant moves with the groundwater and is not retarded by geochemical reactions. The average velocity of the pore water in a porous media was estimated using the following equation:

$$v = -(K\ dh/dx) / n_e$$

where

v = pore water velocity (m [ft]/d)

K = saturated hydraulic conductivity (m [ft]/d)
dh/dx = hydraulic gradient (m [ft]/m [ft])
n_e = effective porosity (decimal)

Travel time (T in days) is then estimated as the distance from the source release to the receptor (D in meter [feet]) divided by the pore-water velocity (v in m [ft]/d). FSAR Table 2.4S.12-17 presents travel times for the four pathways analyzed. The applicant revises and presents this table in a supplemental response to RAI 02.04.12-28 dated November 23, 2009 (ML093310392). The applicant's characterization of the three plausible alternative pathways is shown in Table 2.4S.12-4 of this SER. This table shows the representative value and range for the average linear groundwater velocity and travel time. Estimates of linear groundwater velocity use the high estimate of hydraulic gradient derived from the preconstruction piezometric data.

Table 2.4S.12-4. Pathway Average Linear Velocity and Travel Time (from revised FSAR Table 2.4S12-17 [ML093310392])

Pathway	Avg Velocity (Range) m/d (ft/d)	TRAVEL TIME (year)		
		Representative Value	Low Range Value	High Range Value
Upper Shallow to Southeast	0.04 (0.02-0.11) (0.13 [0.05–0.35])	154	57	400
Lower Shallow to Southeast	0.05 (0.03-0.08) (0.16 [0.11–0.26])	125	77	182
Upper Shallow to Southwest	0.02(0.01-0.04) (0.05 [0.02–0.14])	330	117	821
m/d = meter per day; ft/d= foot per day				

NRC Staff's Technical Evaluation

The staff reviewed FSAR Revision 3, Subsection 2.4S.12.3.2, its proposed revisions, and the revised groundwater model document. The staff's review confirmed that the applicant has addressed the relevant information topics.

In the applicant's supplemental response to RAI 02.04.12-28 (ML092710096), the applicant incorporated updated hydraulic property data into the calculation of advective transport travel times. In the applicant's supplemental response to RAI 02.04.12-38 (ML102450252), the applicant provided technical justification for the hydraulic conductivity data used in their calculations. The staff reviewed these submittals and concurs that the hydraulic conductivity data are justified.

The staff concluded that based on model results and field observations, it is apparent that the hydraulic gradient and travel time estimates for the Upper Shallow Aquifer are conservative because of the likely downward movement of releases from the Upper Shallow Aquifer into the Lower Shallow Aquifer. This closes RAIs 02.04.12-28 and 02.04.12-38 for this subsection.

The applicant's initial estimates of travel time in the Lower Shallow Aquifer were also based on preconstruction hydraulic gradients. Those estimates range from 77 to 182 years with a representative value estimate of 125 years. The applicant provided post-construction simulations and reported a shortest travel time to the site boundary from a postulated Unit 3 release into the Lower Shallow Aquifer of approximately 104 years (see the response to RAI 02.04.12-48 [ML102450252]). Using the site-specific groundwater model (ML110140173), the applicant provides sensitivity cases for the range of saturated hydraulic conductivity and showed a range of travel time from 96 to 127 years (ML103540324). The staff also performed an independent analysis of the influence of a high infiltration rate of 2.54 cm/yr (1 in./yr) and a high backfill hydraulic conductivity of 2.0×10^{-2} cm/s (28.35 in./hr), and simulated a post-construction travel time of 94 years. Because the applicant examined both preconstruction and post-construction conditions, and the simulated range of post-construction travel times lies within the range of simple estimates, the staff accepted the applicant's analysis of advective travel times ranging from 77 to 182 years. This review closes the advective travel times aspect of Open Item 2.4.12-1 in Subsection 2.4S.12.4.12 of this SER.

Groundwater Flow Model

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.3.4, its revision, and the revised groundwater model documentation (ML110140173), the applicant describes a three-dimensional, steady-state, numerical groundwater model developed to better understand preconstruction and post-construction groundwater conditions at the STP site. The model uses the user interface Visual MODFLOW and is based on the USGS-developed MODFLOW 2000 code. This model was calibrated to the September 2008 hydraulic head data. This data set is among the most complete hydraulic head data sets on the Shallow Aquifer, because it contains data from all site characterization wells completed in the summer of 2008. The applicant applied the calibrated model to simulate post-construction conditions, including excavation and backfill at Units 3 and 4, a slurry wall surrounding Units 3 and 4, and postulated releases from Units 3 and 4. Model results provide projections of groundwater hydraulic head and pathways from releases to receptor points.

In response to a series of RAIs issued in April 2010, the applicant improved the performance of the model by incorporating higher resolution topographic data and adjusting the GHB conditions. These modifications to the model, as revealed in the RAI responses (ML102450252), resulted in an improved match to observed conditions or an explanation for the model behavior (i.e., dry cells were explained, wet cells were substantially reduced, and closed contours in the vicinity of drain boundary conditions were substantially eliminated). Calibration metrics of the improved model are not markedly different from those of the previous model, and the applicant described both the previous and improved models as applicable to the site analysis. Post-construction simulations incorporated design information on the structures and structural fill, power block finished grade and backfill cover, slurry wall designs, design of two CFRWs, relocated MDC, and conservative representation of the main cooling reservoir water height. The post-construction cases confirmed that the most plausible pathway from proposed Units 3 and 4 is to the east-southeast in the Lower Shallow Aquifer. The shortest travel time for the plausible pathway was reported as approximately 104 years. The post-construction model was also used to evaluate the water table elevation within the power block and the potential influence of the relief well system on the maximum groundwater elevation. These simulation results supported the applicant's identified maximum groundwater elevation of 8.5 m (28 ft) MSL.

Supplemental responses (ML103540324) to several of the April 2010, RAIs provided additional sensitivity cases on infiltration rate and backfill saturated hydraulic conductivity, and an improved alternative groundwater calibrated to site conditions.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.3.4, its proposed revisions, and the revised groundwater model documentation and simulations.

Responses to the RAIs were received in May and August of 2010, and supplemental responses were received on December 15, 2010. The applicant improved the model by introducing higher resolution topographic data over much of the model and by refining the GHB conditions. The staff understands that these changes involve the model's geometry (i.e., surface and distance to GHB conditions) and the head assigned to the GHB conditions.

Groundwater models such as MODFLOW provide the user with a variety of options for implementing a conceptual model of a site. Based on the staff's review of the improved groundwater model (ML102450252, ML103540324, ML110140173), the staff concluded that the improved model is suitable as the basis for post-construction simulations, especially with regard to the simulation of the Lower Shallow Aquifer. The staff also reviewed the calibrated model (i.e., the improved model used to simulate a main cooling reservoir pool elevation at 12.8 m [42 ft] MSL), the improved model applied to a main cooling reservoir at 14.3 m (47 ft) MSL, and the post-construction simulations (main cooling reservoir at 15.1 m [49.5 ft] MSL) The staff concluded that simulation results for the Lower Shallow Aquifer in the region north of the main cooling reservoir would not change substantially by using a further improved model. The simulation of the preconstruction setting exhibits a gradient from the northwest to the southeast that is somewhat higher than observed in the preconstruction piezometric head data set. The preconstruction and post-construction simulations exhibit piezometric surfaces that support an east-southeastern flow in the Lower Shallow Aquifer and provide an estimate of groundwater travel time to the eastern boundary of the STP site. The staff noted that the calibration data set used by the applicant is among the lowest piezometric head data sets available.

The applicant's response to RAI 02.04.12-47 (ML102450252) addresses the issue of boundary conditions and whether they overly constrain post-construction predictions. The applicant examines a number of alternative GHB conditions for the model and performs a number of sensitivity simulations. The applicant concludes that because the variety of GHBs simulated did not result in "any undue impact" on the water table within the power block, the external boundary conditions are acceptable and do not constrain post-construction predictions. Therefore, RAI 02.04.12-47 is considered closed.

The applicant response to RAI 02.04.12-48 provides (1) sensitivity simulations of post-construction infiltration rates and hydraulic properties of the backfill; (2) simulations showing the influence of structures, slurry walls, and CFRWs; and (3) simulations showing the failure of the relief well system (ML102450252, ML103540324). The post-construction simulations of both items (2) and (3) (ML102450252) included more detail than the previous model, and the staff concurs that the predicted pathways and groundwater levels are representative of the conditions simulated. Simulation of item (1) (ML103540324) included several sensitivity cases including a range of infiltration rates and backfill hydraulic conductivities. The staff concurs that the groundwater model is representative of site conditions and the power block region, and it is sufficient to evaluate groundwater elevation and plausible pathways in response to pre- and post-construction site conditions (e.g., changes in grade, structures, and increased main cooling reservoir level). This review closes the groundwater flow model aspect of RAI 02.04.12-28 and Open Item 2.4.12-1 in Subsection 2.4S.12.4.12 of this SER.

2.4S.12.4.11 Monitoring or Safeguard Requirements

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.4, the applicant describes monitoring of the groundwater system from preconstruction through the startup of the plant. In the ER Revision 3, the applicant states that during the preconstruction period, groundwater levels will be monitored at up to 54 wells that provide observations in both the Upper and Lower Shallow Aquifers. During the detailed design of proposed STP Units 3 and 4, the applicant will review current STP groundwater monitoring programs to identify necessary modifications to incorporate into the monitoring of the proposed units. The review will consider the needed water-level and water-quality measurements for the Deep and Shallow Aquifers, subsidence monitoring in the vicinity of proposed Units 3 and 4, and operational accident monitoring. The applicant will use the reviewed and modified groundwater monitoring programs to monitor groundwater in the Deep and Shallow Aquifers during the construction and preoperational monitoring periods. The applicant will use groundwater monitoring during construction to track changes in groundwater resulting from construction activities including the slurry cut-off wall, CFRWs, and excavation dewatering.

In the ER, the applicant is committed to use best management practices, including well-head protection, to protect the aquifers. The applicant anticipates that the groundwater monitoring required during the operation of proposed Units 3 and 4 will be similar to existing reporting requirements for STP Units 1 and 2 and will be designed and implemented accordingly. However, the applicant acknowledged that the requirements are changing in response to the Nuclear Energy Institute's program to collect groundwater data at commercial nuclear plants. Once construction is complete and the sediment profile has been allowed to rewet, the applicant has committed to the continued evaluation of groundwater levels with the objective of determining whether groundwater level monitoring should continue to ensure that the maximum groundwater level beneath safety-related structures of proposed STP Units 3 and 4 is greater than 61 cm (2 ft) below plant grade at all times (ML082100162).

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.4, and confirmed that the applicant has addressed relevant information. The staff also reviewed ER sections that address groundwater monitoring and radiological environmental operating reports for STP Units 1 and 2 in 2006 and 2007 (Sherwood and Travis, 2007, 2008).

In the FSAR, the applicant describes monitoring and safeguard requirements by stating that current STP monitoring will be evaluated to determine whether any modifications of existing programs are required to adequately monitor the proposed units. Considerations include the following monitoring:

- deep aquifer monitoring of hydraulic head and geochemical quality to detect influences on the groundwater supply or accidental releases

- shallow aquifer monitoring of hydraulic head and geochemical quality to detect changes in flow patterns, potential changes to accident analysis, potential influences on structural stability, and structural integrity

- subsidence monitoring to ensure structural stability

- operational accident monitoring to detect the presence of radionuclides in the environment

The staff reviewed and accepted the applicant's responses to RAI 02.04.12-24 dated July 24, 2008 (ML082100162), which requested clarifications regarding the groundwater monitoring program and its objectives. The applicant's RAI response describes the current monitoring program and commits to review and modify the current plan to address the proposed units. In addition, the RAI response describes the expansion of the current monitoring plan to incorporate industry guidelines for the detection and monitoring of releases of plant-related radionuclides into the groundwater environment. The staff concluded that the applicant's description of how STP will meet the monitoring requirements is appropriate. Therefore RA 02.04.12-24 is considered closed.

The staff noted that the applicant's response to RAI 02.04.12-24 states that STP will perform groundwater-level monitoring "during construction dewatering and rewetting activities" and will evaluate groundwater-level observations after the profile has rewetted to determine whether continued monitoring is warranted or not.

2.4S.12.4.12 Site Characteristics for Subsurface Hydrostatic Loading

Information Submitted by Applicant

In FSAR Subsection 2.4S.12.5, and its proposed revisions, the applicant summarizes the evaluation of hydrostatic loading estimates based on the plant grade and the site characteristic maximum groundwater level. The applicant provides changes to FSAR Subsection 2.4S.12.5 in response to RAI 02.04.12-35 dated September 21, 2009 (ML092710096) and comments on support of the site characteristic in response to RAI 02.04.12-34 (ML092710096) and RAI 02.04.12-49 (ML102450252, ML110450097).

The applicant adopts a site characteristic for a maximum groundwater level of 8.5 m (28 ft) MSL based on field measurements and modeled post-construction results. The applicant states that the post-construction plant grade will be approximately 10.4 m (34 ft) MSL. According to the DCD requirement (i.e., maximum groundwater level is to be greater than 61 cm [2 ft] BGS), the maximum groundwater level shall be no higher than 9.75 m (32 ft) MSL. The applicant evaluates hydrostatic loading by comparing two calculations of hydrostatic load that are (1) based on the DCD requirement, and (2) based on the site characteristic. The applicant states that the site characteristic of 8.5 m (28 ft) MSL satisfies the DCD requirement of 61 cm (2 ft) below plant grade and exhibits a satisfactory hydrostatic pressure.

Support for the selection of a site characteristic of 8.5 m (28 ft) above MSL lies in the field observations of preconstruction groundwater levels inside the power block (ML110450097):

- Over a 34-year period from 1973 through 2007, groundwater levels were below 8.38 m (27.5 ft) MSL in the northern portion of the STP site (ML081890239 and ML082100162).

- Piezometer 602A, the piezometer located nearest to the proposed units, during 1995 through 2006 recorded groundwater elevations below 7.93 m (26 ft) MSL (ML092710096).

- The observation wells within the footprint of the proposed units during 2007 and 2008 show a maximum groundwater elevation of 7.91 m (25.94 ft) MSL (ML092710096).

Support for the selection also lies in the results of post-construction groundwater simulations (ML102450252, ML103540324, ML110140173, ML110450097):

- Post-construction scenarios simulated with the slurry wall in place showed post-construction groundwater levels 30 to 91 cm (1 to 3 ft) lower than preconstruction levels in the Upper Shallow Aquifer in the vicinity of safety-related facilities for Units 3 and 4.

- The post-construction scenarios (including the slurry wall and with the main cooling reservoir at 15.1 m [49.5 ft] MSL) simulated a maximum groundwater level within the proposed power block of about 6.4 m (21 ft) MSL (see Figure 62 in ML110140173).

Additional support for the selection comes from field observations at STP Units 1 and 2 that confirm the creation of a groundwater depression in the region excavated and backfilled. A 0.91- to 1.52-m (3- to 5-ft) depression in the piezometric surface is seen in a May 2006 data set (ML102450252). These observations support the post-construction simulation of the groundwater depressions at STP Units 1 and 2 and proposed Units 3 and 4. A sensitivity case simulated to learn the relationship between the relief wells surrounding the main cooling reservoir examined the case of all relief wells hypothetically removed, and the groundwater elevation within the power block showed a maximum groundwater elevation of approximately 7.86 m (25.8 ft) MSL at the south side of the slurry wall and a simulated maximum groundwater level of less than 7.62 m (25 ft) MSL within the power block.

The applicant concludes that based on historical evidence and post-construction groundwater model results, the maximum post-construction groundwater level at the proposed STP Units 3 and 4 of 8.5 m (28 ft) MSL will not be exceeded, and this site characteristic meets the DCD requirement for the maximum groundwater level.

Based on several factors, the applicant also concludes that "a permanent dewatering system is not anticipated to be a design feature of STP Units 3 & 4." These factors include the following:

- The site characteristic of a maximum groundwater level of 8.5 m (28 ft) MSL.

- Most of the power block surface will be occupied by buildings, structures, and relatively low permeability material (asphalt, concrete). With the exception of buildings and their foundations, the entire power block will be underlain by a low permeability clay layer a minimum of 2 ft thick. Such a power block surface and subsurface will minimize the potential for infiltration and recharge.

- Observations of the STP Units 1 and 2 post-construction water table compared to the pre-construction water table.

With regard to the post-construction power block, roof drains will flow to storm drains. The surface grade within the power block will direct runoff from low-permeability surfaces to storm drains. Storm drains will direct stormwater away from the power block and discharge into surface-water outfalls. With regard to post-construction observations, the effects on the water table from the construction and operation of STP Units 1 and 2 suggest localized changes in the hydraulic head, including communication between the Upper and Lower Shallow Aquifers and an increased drawdown in the Deep Aquifer resulting from production well pumping. The applicant's response to RAI 02.04.12-26 presented a 34-year record of water-level data for the Upper Shallow Aquifer in the vicinity of STP Units 1 and 2. The applicant concludes that groundwater elevations measured before construction of STP Units 1 and 2 have been a good indicator of groundwater elevations after the construction of these units. The applicant assumes that this same concept will apply to STP Units 3 and 4.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.12.2.5 and its proposed revisions and confirmed that the applicant has addressed relevant information.

Based on the results of the probabilistic seismic hazard analysis discussed in FSAR, Section 2.5.2, Vibratory Ground Motion, the staff determined that it is unlikely that ground motion level or associated liquefaction could affect the maximum groundwater level at the STP site. Therefore, a detailed evaluation of this seismically induced groundwater table rising was not performed by the staff as part of this section.

As discussed in FSAR Section 3.8, the DCD site parameter "maximum groundwater level" has been used in conjunction with the most required load combinations—including normal loads combined with earthquake loads, severe winds, or tornado loads. The DCD site parameter "maximum flood level" is exceeded at the STP site during the design-basis flood event, and the maximum groundwater conditions associated with this event are included in the engineering evaluation. The staff's review for subsurface hydrostatic loading is divided into two review topics: (1) the maximum groundwater level under normal conditions and all extreme events excluding the "maximum flood level," and (2) the maximum groundwater level during the event resulting in the "maximum flood level."

With regard to the first topic, the staff independently assessed the maximum groundwater elevation during the post-construction period. The staff obtained an estimate of the maximum groundwater elevation by adding the maximum observed variation in piezometric head (i.e., 1.44 m [4.71 ft]) to the simulated post-construction groundwater within the power block (i.e., 6.4 m [21 ft] MSL). This approach yields an estimate for a maximum groundwater elevation of 7.84 m (25.71 ft) MSL. A second and similar estimate of maximum groundwater elevation is obtained by adding the delta in the model simulation (i.e., the difference between post-construction and preconstruction; negative 0.3 to 0.9 m (1 to 3 ft) implying a groundwater depression), to the observed maximum preconstruction groundwater piezometric head (i.e., 7.91 m [25.94 ft] MSL). This approach yields an estimate for maximum groundwater elevation of approximately 7.62 m (25 ft) MSL. Neither estimate exceeds the proposed site characteristic of 8.5 m (28 ft) MSL.

The staff's review of the site characteristic for the maximum groundwater level also includes consideration of events other than the design-basis flood resulting in surface water inundating the site, (e.g., storm surge, tsunami, dam breach, river flooding, or precipitation conditions resulting in minor flooding). The staff determined that the mechanism that would result in the condition associated with a maximum groundwater level could be any of the above example events. Using soil physics theory to estimate the movement of the wetting front (Jury et al., 1991), the staff estimated that the wetting front would require 28 days to penetrate the 0.6-m-thick (2-ft-thick) clay cap, and years to saturate the upper portion of the natural clay deposit overlying the STP site. The periods of time required for the wetting front to penetrate the clay materials exceeds the duration of any flood event, assuming the clay layers remain intact. Because none of the events would result in scour of the surface profile, substantial areas of the engineered backfill would not be exposed to the surface water. However, in response to RAI 02.04.12-51 (ML103330369), the applicant notes that minor excavations into the clay cap could occur over the life of the plant. The applicant also notes the large extent of the aquifer and the limited extent of future excavations through the clay cap, and concluded that the amount of infiltration would not affect the groundwater level. Although this is dependent on the extent of future excavation, the staff concurred that infiltration into the engineered backfill would be local to such excavations and limited with regard to influence on the overall groundwater level within the power block area.

With regard to long-term precipitation and infiltration the applicant completed a sensitivity simulation (ML103540324) that involved a high estimate of long-term infiltration (2.54 cm/yr or 1 in./yr), and the low estimate of engineered backfill saturated hydraulic conductivity (5.0 x 10^{-4} cm/s [10.6 gpd/ft^2]). This sensitivity simulation was designed to determine the probable maximum groundwater level in the power block area. This simulation resulted in a predicted piezometric head in the Upper Shallow Aquifer well below the site characteristic for maximum groundwater level of 8.5 m (28 ft) MSL. The staff re-simulated and reviewed this case and confirmed the applicant's conclusion.

With regard to the first topic, the maximum groundwater level under normal conditions and all extreme events excluding the "maximum flood level," the staff reviewed the applicant's submittals and performed independent calculations that confirm the applicant's defined site characteristic for maximum groundwater level at 8.5 m (28 ft) MSL.

With regard to the second topic, the maximum groundwater level during the event resulting in the "maximum flood level," the staff's review focused on the design-basis flood, which is the main cooling reservoir breach and flood analysis (see Section 2.4S.4 of this SER). In Section 2.4S.4 of this SER, the staff confirmed that the design-basis flood of 12.2 m (40 ft) MSL is not exceeded. Also in Section 2.4S.4 of this SER, the staff assumed conservatively that the clay cap could be eroded away during the design-basis flood. The erosion of the clay cap would expose the engineered backfill to surface waters for the duration of the design-basis flood. Using soil physics theory (Jury et al., 1991) to estimate wetting front movement under surface water ponded conditions, the staff estimates that infiltration into the engineered backfill would result in saturation of the entire vertical profile from the plant grade to the level of 8.5m (28 ft) MSL. This groundwater conditions is included in the engineering evaluation as discussed in SER Section 3.8.

The staff reviewed FSAR Subsection 2.4S.12.2.5, its proposed revisions, and the applicant's response to RAI 02.04.12-51 and confirmed that the applicant has provided the staff with sufficient information and analyses to close RAI 02.04.12-51 and, with regard to the potential impact on groundwater levels from issues associated with Open Item 2.4.4-1, to close Open Item 2.4.12-1. RAI 02.04.12-51 was issued to obtain information and analyses regarding infiltration during the design-basis flood event. The staff determination resulting in separate reviews of the site characteristic for "maximum groundwater level" and groundwater conditions during the design-basis flood enabled staff to complete the review with the information and analyses provided.

The applicant completed the sensitivity cases described in the August 2010 submittal and submitted a summary of the results on December 15, 2010 (ML103540324). These results were supplemental responses to RAIs 02.04.12-46, 02.04.12-48, and 02.04.12-50, and provided further technical justification for the post-construction subsurface pathways and groundwater level. The applicant submitted the revised groundwater model documentation and the groundwater model input and output files on January 11, 2011. Based on staff's review of applicant's submittals (ML103540324, ML110140173) described above, the responses to these RAIs and the groundwater model are accepted, and this portion of Open Item 2.4.12-1 is closed.

The applicant describes and estimates the potential for the design-basis flood to cause infiltration through the surface and affect (1) the groundwater level within the power block (i.e., could the water table approach or exceed the site characteristic); and (2) the saturation of the upper 2 ft of sediment (i.e., could the subsurface between the plant grade and 2 ft below plant grade become saturated). The applicant considered flood scour and erosion of the power block surfaces and will maintain the surfaces. Based on the staff's review of the applicant's submittals (ML103330369 and ML103630545) with regard to RAI 02.04.12-51 described above, the responses to these RAIs are accepted and this portion of Open Item 2.4.12-1 is closed.

The applicant's RAI responses and associated FSAR revisions demonstrated the strong technical basis for the plausible alternative pathways and their simulation, the site characteristic of the maximum groundwater level, and that the design bases related to groundwater-induced loadings on subsurface portions of safety-related SSCs would not be exceeded under normal conditions and all extreme events excluding the maximum flood level. Accordingly, Open Item 2.4.12-1 is closed.

2.4S.12.5 Post Combined License Activities

There are no post COL activities related to this subsection.

2.4S.12.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the required information related to site groundwater characteristics. The staff confirmed that the applicant has included complete descriptions of the current and future local hydrological conditions, including alternate conceptual models, to demonstrate that the design bases related to groundwater-induced loadings on subsurface portions of safety-related SSCs would not be exceeded. The staff accepted the methodologies used to determine the potential effects of groundwater as documented in safety evaluation reports.

As set forth above, the applicant has presented and substantiated information relative to the groundwater effects important to the design and siting of the proposed plant. The staff reviewed the available information provided and, for the reasons given above, concluded that the identification and consideration of the potential effects of groundwater in the vicinity of the site are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.20(c)(3), 10 CFR 100.23(d)(3), and 10 CFR 100.20(c), with respect to determining the acceptability of the site. The relevant information addressing COL License Information Item 2.32, (i.e., "that the COL applicant analyze the groundwater condition for the specific site"), is adequate and acceptable.

2.4S.12.7 References

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2.4S.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surface Waters

2.4S.13.1 Introduction

This section of the FSAR considers the potential effects of accidental releases from the radwaste management systems that handle liquid effluents generated during normal plant operations. Such releases would have relatively low levels of radioactivity, but they could be large in volume. Normal and severe accidental releases are also considered in the applicant's ER and FSAR Chapter 15. The accidental release of radioactive liquid effluents in groundwater and surface waters is evaluated based on the hydrogeological characteristics of the site. The source term from a postulated accidental release is reviewed under SRP Section 11.2 following the guidance in Branch Technical Position (BTP) 11-6, "Postulated Radioactive Releases Due to Liquid-containing Tank Failures." The source term is determined from a postulated release from a single tank outside of the containment.

This SER section provides an evaluation of the ability of the groundwater and surface-water environment to delay, disperse, dilute, or concentrate liquid effluent, as related to existing or potential future water users.

2.4S.13.2 Summary of Application

In Section 2.4S.13, the applicant addresses the accidental release of radioactive liquid effluents in ground and surface waters. In addition, the applicant provides a site-specific supplement designed to address COL License Information Item 2.21.

COL License Information Item

- COL License Information Item 2.21 — Accidental Release of Liquid Effluents in Ground and Surface Waters

COL license information item directs the applicant to provide site-specific information to address the accidental release of radioactive liquid effluents in ground and surface waters by (1) providing information about the ability of the surface- and subsurface-water environment to disperse, dilute, or concentrate accidental releases; and (2) describing the effects of these releases on existing and known future uses of water resources.

2.4S.13.3 Regulatory Basis

The relevant requirements of the Commission regulations for accidental releases of radioactive liquid effluents in ground and surface waters, and the associated acceptance criteria, are in Section 2.4.13 of NUREG–0800.

The applicable regulatory requirements for liquid effluent pathways for groundwater and surface water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site

- 10 CFR 20, as it relates to effluent concentration limits

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics, with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and the surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The regulatory positions of the following documents are used for the related acceptance criteria:

- BTP 11-6, “Postulated Radioactive Releases Due to Liquid Containing Tank Failures,” provides guidance in assessing a potential release of radioactive liquids after the postulated failure of a tank and its components located outside of the containment, and the impacts of the release of radioactive materials at the nearest potable water supply located in an unrestricted area for direct human consumption or indirectly through animals, crops, and food processing.

- RG 1.113, “Estimating Aquatic Dispersions of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I,” provides guidance in assessing effluent concentration for comparison with10 CFR Part 20, Appendix B effluent concentration limits.

2.4S.13.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.13 of the STP Units 3 and 4 COL FSAR. The staff’s review confirmed that the information in the application addresses the relevant information related to the accidental release of radioactive liquid effluents in groundwater and surface water. The staff’s technical review of this section includes an independent review of the applicant’s information in the FSAR and in the responses to RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff’s evaluation of the technical information presented by the applicant in FSAR Section 2.4S.13.

COL License Information Item

- COL License Information Item 2.21 — Accidental Release of Liquid Effluents in Ground and Surface Waters

Specific information provided by the applicant to address COL Information Item 2.21 includes all material presented in FSAR Section 2.4S.13. The staff reviewed the applicant’s FSAR, its revision, and RAI responses with regard to the accidental release of liquid effluent in groundwater and surface water. The staff’s review of the application is summarized in the following subsection. The staff reviewed the applicant’s submittals using RG 1.206 and the review procedures described in Section 2.4.13 of NUREG–0800.

2.4S.13.4.1 Direct Release to Groundwater

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1, and its proposed revisions, the applicant provides an analysis of the postulated accidental liquid release into groundwater at the STP site. The applicant includes in the pathway analysis the processes of advection, dispersion, retardation, and decay. The applicant analyzes the plausible alternative pathways developed and presented in FSAR Section 2.4S.12. The analysis was applied to the plausible alternative pathways in two stages. The first stage considers non-retarded groundwater travel time (advection) and decay only to eliminate the majority of radionuclides that have relatively short half-lives. The second stage considers advection, dispersion, retardation, and decay to evaluate all radionuclides that pass the first stage. Reactions with the sediments can reduce the radionuclide concentration through cation/anion exchange, complexation, oxidation-reduction, and surface sorption. The applicant chose to simulate the combination of geochemical reactions with the linear sorption isotherm model.

NRC Staff's Technical Evaluation.

The staff reviewed the introductory material and FSAR Subsection 2.4S.13.1. The staff's review confirmed that the applicant has addressed relevant information. The applicant quoted from ABWR DCD Tier 2, Subsection 15.7.3.3, and the staff confirmed the following statement regarding a postulated radioactive release due to liquid radwaste tank failure: "(t)he liquid pathway is not considered due to the mitigative capabilities of the Radwaste Building." Furthermore, the staff noted in ABWR DCD Tier 2, Subsection 12.2.1.2.10, the following statement: "potential releases in the Radwaste Building will be contained by filtering the Radwaste Building atmosphere and sealing any water releases in the building, which is steel-lined to prevent any potential water releases." The applicant quotes from ABWR DCD Tier 2, Subsection 15.7.3.1, and NRC confirmed that "(t)he probability of a complete tank release is considered low enough to warrant this event as a limiting fault." However, for the purpose of conservatism, the applicant concludes and NRC confirmed that the postulated rupture of a radwaste tank in the ABWR radwaste building is considered limiting for the analysis of accidental releases of radioactive liquid effluents in groundwater and surface water.

The staff accepted the applicant's statements describing the groundwater pathway as being conservatively represented by the processes of advection, dispersion, retardation, and decay. The staff also accepted that geochemical reactions between the radioactive liquid effluent and the aquifer matrix could include cation/anion exchange, complexation, oxidation-reduction reactions, and adsorption on surfaces. And the staff acknowledged that decay can be significant, especially for short-lived radionuclides. The staff concluded that the applicant's general description of the direct release into groundwater is accurate.

The staff reviewed the statements of this section and the description of the approach in the response to RAI 02.04.13-1, which requested a description of the process followed to identify plausible alternative pathways. The latter was incorporated by the applicant into FSAR Subsection 2.4S.12.1.1. The staff found the statements of the process of data review and assimilation to formulate plausible alternative pathways and conceptual models satisfactory. Therefore, RAI 02.04.13-1 is considered closed.

2.4S.13.4.2 Accident Scenario

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.1, the applicant-postulates that the release into groundwater and surface water is from a liquid radwaste tank rupture in the radwaste building. The applicant's review of radioactive sources concludes that the low conductivity waste (LCW) collector tank would be the best choice based on its having the greatest concentrations of radioisotopes. There are four LCW collection tanks, each with a volume of 140 m^3 (36,984 gal) (ABWR DCD Tier 2, Section 11.2, Table 11.2-4). Based on BTP 11-6 guidance, the postulated rupture of one LCW collector tank is assumed to release 80 percent of its liquid volume (112 m^3 [29,587 gal]) into the groundwater environment. The release into the groundwater is assumed to reach the aquifer without being diluted. The radionuclide concentrations assigned to the tank rupture in FSAR Table 2.4S.13-1, are the highest radionuclide concentrations from either the LCW collector tank concentrations or the reactor coolant concentrations (DCD Tier 2, ABWR Revision 4, Section 11.1).

The applicant notes that the radwaste building includes numerous components that make a release into groundwater from a radwaste tank in the building unlikely. The building design includes a basemat and walls to a height needed to retain spilled liquids, and the rooms containing the LCW tanks are steel lined to a height capable of retaining the contents of the tank. Furthermore, the rooms are equipped with alarmed tank-level monitoring and a sump collection system to collect any leakage. Part 7 of the Departures Report STD DEP 11.2-1 states that a release into the groundwater is not considered credible.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.1. The staff's review confirmed that the applicant has addressed relevant information. The staff reviewed the postulated release into groundwater and surface water and found the postulated liquid radwaste tank rupture in the radwaste building to be consistent with the ABWR DCD information and BTP 11–6. The staff reviewed and accepted the radionuclide concentrations reported in FSAR Table 2.4S.13-1, as the highest from either the LCW collector tank or the reactor coolant concentrations (DCD Tier 2, Revision 4, Section 11.1).

2.4S.13.4.3 Conceptual Model

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.2, and its proposed revisions, the applicant describes the conceptual model(s) used to evaluate the plausible groundwater pathways that an accidental release of radioactive liquid effluent could follow at the proposed STP site. The model is based on the hydrogeological data and interpretations in FSAR Section 2.4S.12. In brief, there are two aquifers underlying the STP site: the Shallow Aquifer and the Deep Aquifer. They are separated by a 30 to 46 m (100- to 150-ft) thick deposit of clay and silt. The Shallow Aquifer can be subdivided into an Upper and Lower Shallow Aquifer that are separated by an approximately 6.1 m (20 ft) thick deposit of clay and silt.

In FSAR Section 2.4S.12, the applicant concludes that groundwater flow is predominantly to the east-southeast from the reactor location toward the Colorado River in the Upper and Lower

Shallow Aquifer, with some potential for flow toward the southwest and the western side of the STP site in the Upper Shallow Aquifer. Along the predominant pathway to the southeast, the applicant selected three exposure points as plausible: (1) the east site boundary where the Upper Shallow Aquifer could be intercepted by an unnamed tributary and where the Upper and Lower Shallow Aquifer could release into a hypothetical offsite water-supply well; (2) an existing offsite water well (number 2004120846); and (3) the Colorado River. Kelly Lake is also identified as a plausible receptor location. All east and southeast directed pathways, including Kelly Lake, are conservatively represented by a pathway intercepted by a hypothetical water-supply well assumed to be located at the eastern site boundary. The applicant also admitted a fourth pathway, a west-southwest pathway in the Upper Shallow Aquifer from proposed Unit 4 to the western site boundary, where a similar hypothetical water-supply well is assumed to be located.

Based on the site characterization data showing that the hydraulic head of the Upper Shallow Aquifer is higher than that in the Lower Shallow Aquifer, the applicant has concluded that a downward hydraulic gradient will likely force radioactive liquid effluents released into the Upper Shallow Aquifer downward into the Lower Shallow Aquifer. The applicant also noted that the third exposure point to the southeast, which releases into the Colorado River, would represent a combined groundwater and surface-water pathway that could be further analyzed using the minimum 7-day low-flow rate of the Colorado River, approximately 14.2 Lps (0.5 cfs). This low-flow value is based on Colorado River flow data from 1948 through 2006.

The applicant considers and eliminates several exposure points, pathways, or transport processes (i.e., the applicant found them not to be plausible), as follows:

- exposure at or attributed to the relief wells surrounding the main cooling reservoir
- exposure from a Deep Aquifer pathway
- exposure at the western side of the STP site from a southwest groundwater pathway in the Lower Shallow Aquifer
- a pathway in the Upper Shallow Aquifer related to the thermal buoyancy during the release
- enhanced transport because of the presence of chelating agents

The applicant finds the exposure points, pathways, or transport processes described above not to be plausible for the following reasons, respectively:

- Groundwater flow is from the main cooling reservoir past the relief wells and toward STP Units 3 and 4. Therefore, relief wells will not be exposure points.
- The Deep Aquifer is separated from the Shallow Aquifer by a low-conductivity confining unit of at least 30 m (100 ft) of clay and silt, and piezometric level data show that groundwater in the Deep Aquifer underlying the STP site is drawn to the STP production wells, making an offsite release unlikely.
- The applicant concludes from potentiometric and hydraulic conductivity data that groundwater flow to the southwest in the Lower Shallow Aquifer, if it exists, is seasonal and is impeded by low-conductivity materials. The applicant also considered post-

construction simulations of the site in reaching this conclusion (ML102450252). Despite the appearance of a small mound in the Lower Shallow Aquifer beneath the proposed units in simulations, groundwater flows to the east-southeast toward the site boundary.

- The applicant's analysis of thermal buoyancy concludes that the temperature delta (i.e., the temperature difference between the mixture of spilled radwaste and ambient groundwater) could be 2.5°C (4.5 °F) (or 4.9°C [8.8 °F] in a sensitivity case [ML092710096]), and this delta temperature "would not likely cause buoyancy." A release from the radwaste building would occur within the backfilled excavation that is expected to exhibit a downward hydraulic gradient from the Upper toward the Lower Shallow Aquifer.

- The applicant evaluated conditions that could lead to chelating agents enhancing migration in aquifers and found that conditions at the STP site made it unlikely that the complexation of radionuclides by organic chelating agents would significantly influence groundwater pathways.

The conceptual model of the groundwater pathway is for groundwater from postulated releases at STP Units 3 and 4 to move to the east-southeast in the Upper and Lower Shallow Aquifer to a conservative exposure point represented by a hypothetical water-supply well along this pathway and located on the eastern boundary of the site. A groundwater pathway to the west-southwest from proposed Unit 4 to the western site boundary is also assumed to be intercepted by a water-supply well.

NRC Staff's Technical Evaluation.

The staff reviewed FSAR Subsection 2.4S.13.1.2 and the proposed revisions. In the FSAR the applicant adopted the east-southeast directed groundwater pathway within the Upper and Lower Shallow Aquifer and a west-southwest directed groundwater pathway within the Upper Shallow Aquifer, as the plausible pathways for an accidental radioactive liquid effluent release into groundwater. The applicant considered but eliminated the following list of alternative groundwater pathways, exposure points and transport processes:

- exposure at or attributed to the relief wells
- exposure from a Deep Aquifer pathway
- exposure at the western boundary of the STP site from the Lower Shallow Aquifer
- a pathway related to thermal buoyancy
- enhanced transport because of the presence of chelating agents

The staff reviewed and accepts the east-southeast and west-southwest pathways described by the applicant as plausible groundwater pathways. Site characterization data and simulations are sufficient to support this conclusion. With regard to the alternative groundwater pathways, exposure points, and transport processes eliminated by the applicant, the staff reviewed each alternative and concluded the following:

- The staff reviewed the applicant's supplemental information in the responses to RAIs. These RAIs discussed the relief wells (ML081960070) and the potentiometric surface in the vicinity of the main cooling reservoir. The staff concluded that groundwater moves away from the main cooling reservoir and into the Upper Shallow Aquifer (ML092710096 and ML093310392), and the staff accepted the elimination of relief wells as an exposure point.

- The staff reviewed the potential for exposure via the Deep Aquifer and acknowledged the substantial separation between the Lower Shallow Aquifer and the Deep Aquifer, and the potentiometric data demonstrating that flow within the Deep Aquifer beneath the STP site is toward the STP production wells. The applicant acknowledges that the increase in groundwater production consistent with the construction and operation of STP Units 3 and 4 will create lower potentiometric levels in the Deep Aquifer, a larger cone of depression, and an expanded area of lower potentiometric head over most of the northern portion of the STP site. The staff concluded that the vertical hydraulic gradient will increase, thereby causing a shorter travel time through the 30- to 46-m (100- to 150-ft) thick confining strata that separate the Shallow and Deep Aquifer. However, any release into the Deep Aquifer would be drawn into STP production wells. The staff also accepted the concept that releases into the Shallow Aquifer will likely travel in and discharge from the Shallow Aquifer to adjacent surface waters, rather than move into the Deep Aquifer.

- The staff reviewed the alternative pathway in the Lower Shallow Aquifer to the southwest of STP Units 3 and 4. The staff found that a southwest pathway and exposure point on the western site boundary is not plausible in the Lower Shallow Aquifer for the following reasons:

  - The evaluation of hydraulic properties in the region to the west-southwest of proposed STP Units 3 and 4 and the evaluation of the continuity of geohydrologic units in this region of the Lower Shallow Aquifer suggest that groundwater movement from the proposed units will be less likely to occur to the west-southwest than to the east-southeast.

  - Potentiometric data for the Lower Shallow Aquifer in the vicinity of STP Units 1 and 2 show a flattening of the potentiometric surface and perhaps a very localized and low groundwater mound. The STP Units 3 and 4 excavations will create communication between the Upper and Lower Shallow Aquifers and will likely create a higher potentiometric surface in the Lower Shallow Aquifer at the postulated source release point. Simulations of the mound underlying Units 3 and 4 do not suggest that a west-southwest pathway will develop. Based on current information, the staff acknowledged that a Lower Shallow Aquifer pathway will likely move beneath the main cooling reservoir before crossing the site boundary to the east.

  - The groundwater model and pathway analyses upon which the applicant based the plausible pathway decision were revised and provided to the staff. Supplemental RAI responses (ML103540324) were provided to the NRC on December 15, 2010. The revised groundwater model document (ML110140173) was provided to the NRC on January 11, 2011. The results of an alternative conceptual model using a spatially varying hydraulic conductivity distribution and the results of several sensitivity cases support elimination of a west-southwest pathway in the Lower Shallow Aquifer from the power block to the site boundary or LRS. These RAI responses and model documentation were reviewed by the staff and resulted in closing Open Item 2.4.12-1. (see Section 2.4S.12 of this SER).

- The staff reviewed the applicant's analysis of thermal buoyancy. The staff noted that the inclusion in the analysis of release and transport in the Upper Shallow Aquifer to the east-southeast and west-southwest made a further analysis of thermal buoyancy unnecessary, because the buoyancy-related pathway in the Upper Shallow Aquifer was included by the applicant.

- The staff reviewed the applicant's responses to RAI 02.04.12-17 (ML082270381) and RAI 02.04.13-7 (ML081970231). The staff accepted the applicant's conclusion that based on the unlikely release of chelating agents, substantial dilution by groundwater, and the abundant source of competing cation clay and silt minerals, there will be a minimal potential for the enhancement of radionuclide migration due to the presence of chelating agents.

The staff's review of the applicant's information and data supporting the conceptual model topic confirmed that the applicant has addressed relevant information. The east-southeast and west-southwest directed pathways in the Upper Shallow Aquifer and the east-southeast directed pathway in the Lower Shallow Aquifer are accepted as plausible pathways with multiple exposure points. The applicant provided additional information relevant to the west-southwest pathway on December 15, 2010 (ML103540324), in response to RAIs issued in April 2010. The staff's reviews of these supplemental RAI responses and the revised groundwater model documentation (ML110140173) resulted in closing Open Item 2.4.12-1. The staff concluded that the west-southwest directed pathway in the Lower Shallow Aquifer can be excluded.

2.4S.13.4.4 Analysis of Accidental Releases to Groundwater

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.3, and its proposed revisions, the applicant describes the application of an approach to estimating radioactive contaminant concentrations in the groundwater pathway resulting from the postulated release from an LCW collection tank into groundwater surrounding the radwaste building. The applicant presents a two-step, one-dimensional calculation. In this calculation, the applicant considers parent and progeny radionuclides that are expected to be present in the LCW tanks and groundwater pathways. First, a calculation using the groundwater travel time (i.e., unretarded travel time) and decay is performed as a simple screen to eliminate radioisotopes that will have little effect on the public because they have short half-lives relative to the groundwater travel time. Second, a calculation using the transport processes of advection, dispersion, retardation, and decay is performed to provide a more realistic, yet still conservative, analysis of radioisotope concentrations at exposure points.

The applicant provides progeny radioisotopes in FSAR Figure 2.4S.13-1, and includes members of each decay chain identified by International Commission on Radiation Protection Publication 38 (ICRP, 1983) to be considered in dose calculations. The results of the two-step calculation process are compared to the maximum permissible concentrations (i.e., the effluent concentration limits or ECLs) found in 10 CFR Part 20, Appendix B, Table 2, Column 2. The applicant applied progressively more realistic and less conservative assumptions to show compliance in the second step, considering only those radionuclides for which the results of the first step produced radioisotope concentrations greater than or equal to 1 percent of the ECL.

The first step is a screening calculation to identify radioisotopes to be further analyzed, and it assumes all the radionuclides migrate at the same rate as the groundwater. This assumption allows the Bateman equations as given in FSAR Equation 2.4S.13-8, -9, and -10, and in Appendix B of NUREG/CR–5512, Vol. 1 (Kennedy and Strenge, 1992) to be applied to the parent and first and second progeny.

The second step uses a standard equation and solution for one-dimensional transport along a groundwater pathline that includes the processes of advection, dispersion, retardation, and radioactive decay. The analytical solution is taken from Water Resources Monograph 10 published by the American Geophysical Union (Javandel et al., 1984).

The applicant performed the first step screening calculation on the groundwater pathway directed to the east-southeast of STP Unit 3 and on the exposure point at the eastern site boundary. Because all other east-southeast exposure points are on the same pathway but are farther from the source, the results of an analysis of the eastern site boundary exposure point are conservative for all exposure points considered. The applicant used both the representative average linear groundwater velocity reported in FSAR Section 2.4S.12 (see FSAR Tables 2.4S.12-14 and 2.4S.12-17), and the high estimate of linear groundwater velocity in the calculations. For the east-southeasterly pathway from Unit 3, the results of the screening analysis using the representative linear groundwater velocity identified radionuclides Ni-63, Sr-90, Y-90, Cs-137, and Pu-239 as analytes for further analysis. An analysis using the higher linear groundwater velocity and the lower travel time identified these radionuclides plus H-3 and Co-60 as analytes for further analysis. The analysis of the west-southwesterly pathway from Unit 4 for the representative linear groundwater velocity identified radionuclides Ni-63, Sr-90, Cs-137, and Pu-239 as analytes for further analysis. The use of the higher linear groundwater velocity identified H-3 and Y-90 as additional radionuclides for further analysis.

The second calculation step yields a more realistic and less conservative estimate of radionuclide concentration. Distribution coefficients for Co, Ni, Sr, Cs, and Pu were taken from a site-specific study, and the geometric mean of the lognormal distribution was used in the analysis as a “best” representation the geochemistry of Shallow Aquifer sediments. For the analyte tritium (H-3), there is no adsorption and its distribution coefficient was assigned a zero value. For the analyte yttrium (Y-90), there are no site-specific measurements. Its adsorption was assumed to be similar to that of scandium, an element adjacent to yttrium in the periodic table and estimated from literature values for scandium. For the purpose of conservatism, distribution coefficient values taken from the literature used the lowest 10th percentile probability value in the analysis. For all analytes analyzed in the Upper Shallow Aquifer pathway the dispersivity, total porosity, effective porosity, and bulk density values used in the analysis were 15.3 m (50.3 ft), 0.41, 0.33, and 1.58 g/cc (98.6 lb/ft^3), respectively. For all analytes analyzed in the Lower Shallow Aquifer pathway, these values were 15.3 m (50.3 ft), 0.39, 0.31, and 1.63 g/cc (101.8 lb/ft^3), respectively. The second calculation step for representative estimates of linear groundwater velocity and for both east and west directed pathways found no effluent concentration limit (ECL) violations and no sum of fraction violations at the eastern and western site boundary.

NRC Staff’s Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.3, and its proposed revisions. The staff confirmed that the applicant has addressed relevant information. The pathway analysis of an accidental release into groundwater for the STP site described by the applicant in FSAR Subsection 2.4S.13.1.3, focuses on three pathways: an east-southeast directed Upper Shallow Aquifer pathway, an east-southeast directed Lower Shallow Aquifer pathway, and a west-southwest directed Upper Shallow Aquifer pathway. Each is conservatively represented by exposure via a water-supply well at the respective site boundary, east and west. The staff reviewed the two-step analysis methodology presented by the applicant and reviewed its application to the three aquifer pathways.

The staff noted that the full decay chains do not appear to have been analyzed in the first step of the analysis. For example, the results of the analysis in revised FSAR Tables 2.4S.13-2A and 2.4S.13-2B (ML093310392) do not include the long-lived isotopes resulting from the Mo-99 and Te-129m decay chains, which include Tc-99 and I-129 shown in FSAR Figure 2.4S.13. The applicant's response to RAI 02.04.13-8 (ML082270381) states that the decay chains were truncated at a "progeny member where incremental dose from the total energy from all radiation emitted over a 100-year period is not significant." The staff independently confirmed the applicant's truncation process and found that the complete conversion of Mo-99 to Tc-99 and of Te-129m to I-129 yields a negligible dose. Therefore, RAI 02.04.13-8 is considered closed.

The staff reviewed the first and second step of the groundwater pathway analysis and accepts the methodology and the hydraulic and geochemical data applied. The staff performed an independent calculation for both steps using the Bateman equations (Kennedy and Strenge, 1992) and a more conservative approach omitting the dispersion phenomena in the second step. Based on a representative analysis of the pathway with the shortest travel-time-the east-southeast pathway in the Upper Shallow Aquifer from Unit 3 to a hypothetical water-supply well on the eastern boundary of the site—the first step of the screening analysis correctly identified analytes for further analysis. The second step, which included adsorption phenomena, showed no ECL violations and no sum of fractions violations.

The analysis of the radioactive liquid effluent transport through the groundwater pathway relies on all plausible pathways being identified for analysis. Supplemental responses to RAI 02.04.12-46 on the spatial bias in the model results, RAI 02.04.12-48, Part 1 on post-construction infiltration, and RAI 02.04.12-50 on groundwater mounding in the Lower Shallow Aquifer, were received on December 15, 2010 (ML103540324) and the revised groundwater model document was received on January 11, 2011 (ML110140173). The staff's review of these submittals fully supports the pathways previously identified by the applicant, supports the applicant's initial position of dismissing a west-southwest directed pathway in the Lower Shallow Aquifer, and supports closure of Open Item 2.4.12-1.

2.4S.13.4.5 Compliance with 10 CFR Part 20

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.1.4, and its proposed revisions, the applicant describes the comparison of the analysis results to the requirements of 10 CFR Part 20. The applicant's analysis evaluated the postulated accidental release of radioactive liquid effluents from the LCW collection tanks in the radwaste building into the Upper and Lower Shallow Aquifer pathway to the east-southeast of STP Unit 3, and the Upper Shallow Aquifer pathway to the west-southwest of STP Unit 4. This analysis shows that each of the radioactive analytes is below its respective ECL at the plausible and conservative exposure point (i.e., the hypothetical water-supply well at the eastern or western site boundary). The applicant has taken the sum of fractions approach and using the estimated radionuclide concentrations, has shown that the sum of fractions is below 1 for each pathway.

The applicant also performs a sensitivity analysis using the range of average linear velocity (see FSAR Table 2.4S.12-17; note that this table was revised in ML093310392) and the range of distribution coefficients (FSAR Table 2.4S.13-3). The applicant pairs relatively extreme conditions of maximum groundwater velocity and minimum distribution coefficients (rapid migration) and minimum groundwater velocity and the maximum distribution coefficient (slow migration). Where site-specific distribution coefficients were not available, the applicant applies the upper and lower bounds of the 95 percent confidence interval from literature values (ML092610376). Results of the sensitivity analysis showed that no exceedance of ECLs occurs for the case of rapid migration, the limiting case. The applicant notes that the variability of the geologic depositional environment underlying the STP site—and the resulting discontinuous fine-grained mixtures of sediment—suggest that average and geometric mean values of properties best represent the STP site.

The applicant considers the analysis conservative for the following reasons:

- The analysis omits the processes of dilution during release and diffusion during transport, and both would cause concentrations to be reduced.

- The analysis assumes that no mitigative measures are taken to remove the radioactive source or to reduce radioactive concentrations in the groundwater.

- Credit is not taken for design elements of the radwaste building and the overall radwaste system that should prevent the release from occurring.

- Because the radwaste building foundation is below the water table, the release from a leaking exterior wall would require the building to first fill with groundwater to the water table height. Until that time, groundwater flow would be inward and the release could not occur. The time required would provide an opportunity for mitigative measures.

The applicant concludes that the STP site groundwater pathway yielded an analysis that demonstrated compliance with 10 CFR Part 20, Appendix B, Table 2. Compliance was demonstrated for both individual radioisotopes and through the sum of fractions, for mixtures of radioisotopes.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.1.4 and its proposed revisions. The staff confirmed that the applicant has addressed relevant information. The staff reviewed the representative and sensitivity cases presented by the applicant for the accidental release of radioactive liquid effluent into the groundwater. The representative case incorporates a one-dimensional model, and most properties are representative (i.e., the geometric mean of saturated hydraulic conductivity and the arithmetic mean of porosity). The hydraulic gradient is estimated as the high end of the observed range from the preconstruction piezometric surfaces. The sensitivity analysis performed by the applicant used the high end of the range of saturated hydraulic conductivities and the low end of the range of distribution coefficients to simulate the minimum travel-time case. This case also used the high estimate of the hydraulic gradient based on preconstruction data.

Simulations using the site groundwater model included the post-construction case of the confining layer between the Upper and Lower Shallow Aquifer being excavated and replaced with engineered backfill. Post-construction settings also examined the influence of a higher main cooling reservoir elevation. The groundwater model simulations estimated travel times to the eastern site boundary within the range predicted by the one-dimensional model.

The staff performed an independent calculation to review groundwater concentrations and the sum of fractions calculated by the applicant. The staff concurs that the results of the representative case and the minimum travel-time sensitivity case presented by the applicant comply with 10 CFR Part 20. The staff's review of the applicant's responses to RAIs associated with Open Item 2.4.12-1 resulted in closing the open item. There were no revisions to FSAR Section 2.4S.13.

2.4S.13.4.6 Direct Releases to Surface Waters

Information Submitted by Applicant

In FSAR Subsection 2.4S.13.2, the applicant describes the credibility of flood events to result in a surface-water release from the radwaste building. The applicant notes that all tanks containing radioactive liquid effluents for STP Units 3 and 4 are inside the radwaste building, and there are no outdoor tanks in the liquid waste management system (LWMS). Notwithstanding the numerous design features of the radwaste building and radwaste system that make a release unlikely, the applicant determined that the most plausible accident scenario that could result in a release into surface water is a rapid and catastrophic flood such as a breach of the main cooling reservoir embankment (i.e., the design-basis flood), coinciding with leakage from the indoor tanks on the basement level of the radwaste building, (i.e., not unlike that described in FSAR Subsection 2.4S.13.1.1). Both of these events, (i.e., the design-basis flood and tank leakage within the radwaste building) are unlikely extreme events.

The applicant considers other external flood events to be slow-moving events that would allow ample warning and time to initiate actions that would mitigate the potential effects from flooding. Therefore, the applicant determined that none of the other external flood events was credible for use in the scenario of a direct release into surface water.

The applicant summarizes the effect of a coupled main cooling reservoir breach flood and radwaste building release event as follows:

- This magnitude of flooding would disperse and dilute the radionuclide concentration.
- There are no known users of the Colorado River or the LRS water downstream of the STP site.
- Therefore, no surface-water users would be affected.

In response to RAI 02.04.13-13 dated September 16, 2009 (ML092610376), the applicant uses main cooling reservoir breach flood and radwaste building release volumes and LCW radioisotope concentrations to quantify the level of radioactive contamination from the direct release of an accidental radioactive liquid effluent into surface waters. Using the 10 CFR Part 20 ECLs, the applicant demonstrated that the result of a main cooling reservoir breach flood and a coincident release from the radwaste building is a small fraction of the 10 CFR Part 20 limits.

NRC Staff's Technical Evaluation

The staff reviewed FSAR Subsection 2.4S.13.2, and its proposed revisions (ML 092610376). The applicant has stated there are no outdoor tanks in the LWMS; therefore, any accidental release of radioactive liquid effluent would come from a tank within a building inside the power block. The postulated release from the LCW collector tank in the radwaste building is such a release (see Subsection 2.4S.13.4.2 of this SER) and it represents an unlikely extreme event. All events resulting in surface water inundating all or a portion of the STP site are also unlikely extreme events, (e.g., storm surge, tsunami, dam breach, river flooding). Therefore, any direct release to surface water from an accidental release of radioactive liquid effluent would result from the combination of two unlikely extreme events. The staff determined that unlikely extreme events should not be combined. Therefore, there is no scenario for a direct release into surface water.

The postulated release to groundwater, which was discussed in the preceding subsections of this section of the SER, would continue to move past the site boundary and eventually release to surface water, (e.g., the Colorado River). This represents an indirect release to surface water. However, such a release would experience additional environmental delay and dispersal, and, in the case of adsorbed contaminants, additional retardation and decay of the liquid effluent before being released to surface water. Accordingly, such an indirect accidental release of radioactive liquid effluent to surface water would involve lower concentrations than previously discussed and found acceptable.

In summary, the applicant has included sufficient relevant information to enable the staff's review of a direct release to surface water. The staff reviewed FSAR Section 2.4S.13, its proposed revisions, and the RAI responses. The staff concluded that there is no scenario for a direct release to surface water.

2.4S.13.5 Post Combined License Activities

There are no post-COL activities related to this subsection.

2.4S.13.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the relevant information related to the effect of accidental releases of radioactive liquid effluent in ground and surface waters. As set forth above, the applicant has presented and substantiated information relative to the accidental releases of radioactive liquid effluent in ground and surface waters important to the design and siting of the proposed nuclear power plant.

The staff reviewed the information in the application addressing COL License Information Item 2.21. For the reasons given above, the staff concluded that the identification and consideration of the potential effects of accidental releases of radioactive liquid effluents in ground and surface waters on existing users and known and likely future users of ground and surface water resources in the vicinity of the site are acceptable and meet the requirements of 10 CFR 52.79, 10 CFR 100.23(d)(3), and 10 CFR 100.20(c) with respect to determining the acceptability of the site. Therefore, the information addressing COL License Information Item 2.21 is adequate and acceptable.

2.4S.13.7 References

10 CFR Part 20, Appendix B. Title 10 of the *Code of Federal Regulations*., Energy, Part 20 “Standards for Protection Against Radiation; Appendix B to Part 20, Annual Limits on Intake (ALIs) and Derived Air Concentrations (DACs) of Radionuclides for Occupational Exposure; Effluent Concentrations; Concentrations for Release to Sewerage”

International Commission on Radiation Protection (ICRP), 1983. “Radionuclide Transformations – Energy and Intensity Emissions, ICRP Publication 38, -13”

Javandel, I, C. Doughty, and C-F Tsang, 1984. Groundwater Transport: Handbook of Mathematical Models. Water Resources Monograph Series #10, American Geophysical Union, Washington, D.C.

Kennedy, W.E. and D.L. Strenge, 1992. Residual Radioactive Contamination From Decommissioning. NUREG/CR-5512, Vol. 1, U.S. Nuclear Regulatory Commission, Washington, D.C.

Branch Technical Position 11-6, “Postulated Radioactive Releases Due to Liquid-Containing Tank Failures,”, NUREG–0800, U.S. Nuclear Regulatory Commission, March 2007, Washington, D.C.

2.4S.14 Technical Specifications and Emergency Operation Requirements

2.4S.14.1 Introduction

This section of the FSAR describes the technical specifications and emergency operation requirements as necessary. The requirements described implement protection against floods for safety-related facilities to ensure that an adequate supply of water for shutdown and cool-down purposes is available.

This SER section provides an evaluation of the following specific areas: (1) controlling hydrological events, as determined in previous hydrology sections of the FSAR, to identify bases for emergency actions required during these events; (2) the amount of time available to initiate and complete emergency procedures before the onset of conditions while controlling hydrological events that may prevent such action; (3) reviewing technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications; (4) potential effects of seismic and non-seismic information on the postulated technical specifications and emergency operations for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4S.14.2 Summary of Application

In Section 2.4S.14 the applicant addresses technical specifications and emergency operation requirements. In this section, the applicant provides site-specific supplemental information to address COL License Information Item 2.22 identified in DCD Tier 2, Revision 4, Section 2.3.

The applicant addressed the information as follows:

COL License Information Item

- COL License Information Item 2.22 Technical Specifications and Emergency Operation Requirement

COL License Information Item 2.20 requires the COL applicants to provide site-specific information related to flood-protection measures for STP Units 3 and 4 safety-related facilities and provisions to ensure that an adequate water supply is available to shut down and cool the reactor. The applicant provides supplemental information to establish technical specifications (TS) and emergency operating procedures (EOPs) to ensure these measures. The applicant commits (COM 2.4S-1) that appropriate EOPs will include applicable provisions for the main cooling reservoir that are similar to those provided for STP Units 1 and 2, before fuel loading.

2.4S.14.3 Regulatory Basis

The relevant requirements of the Commission regulations for the TS and emergency operation requirements, and the associated acceptance criteria, are in Section 2.4.14 of NUREG-0800.

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 50.36, as it relates to identifying technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications.

In addition, the staff used the regulatory positions of the following regulatory guides for the identified acceptance criteria:

- RG 1.59, as supplemented by the current best practices, provides guidance for developing the hydrometeorological design bases.

- RG 1.102 describes acceptable flood protection to prevent the safety-related facilities from being adversely affected.

2.4S.14.4 Technical Evaluation

The staff reviewed the information in Section 2.4S.14 of the STP Units 3 and 4 COL FSAR. The staff's review confirmed that the information in the application addresses the relevant information related to the TS and EOPs. The staff's technical review of this section included an independent review of the applicant's information in the FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in FSAR Section 2.4S.14.

COL License Information Item

- COL License Information Item 2.22 — Technical Specifications and Emergency Operation Requirement

The staff reviewed the applicant's supplemental information on the TS and EOPs. The staff's review of the application is summarized below:

Information Submitted by Applicant

The applicant states that safe plant operations for STP Units 3 and 4 will not be affected by floodwater elevations, because all required systems and equipment are protected against the design-basis flood and will therefore remain operational during such an event.

The applicant states that the design-basis flood elevation of 12.2 m (40 ft) MSL is the result of a postulated failure of the main cooling reservoir embankment. Site grades in the power block area of STP Units 3 and 4 range from 9.8 to 11.2 m (32 to 36.6 ft) MSL, and the top of the concrete floor elevation of the structures located within the power block area is at 10.7 m (35 ft) MSL.

The applicant states that the structural and watertight flood-protection measures are applied to any STP Units 3 and 4 facilities that have an open passageway to any safety-related facility. For all facilities, watertight doors and hatches that are located below 12.2 m (40 ft) MSL will remain closed and under administrative control. Therefore, the applicant concludes that no EOPs or plant TS are required to implement flood protection for STP Units 3 and 4.

The applicant states that with the exception of the main cooling reservoir embankment breach, flooding at the STP site is not a sudden event. During precipitation-induced flooding, the rise in river water elevation is gradual and slow. The approach of a hurricane can be forecasted and its trajectory can be tracked. The applicant estimates the shortest warning time during a postulated upstream dam failure on the Colorado River as 58 hours in FSAR Section 2.4S.4. Consequently, the applicant concludes that adequate time is available to implement remedial or preventive measures for non-safety-related facilities.

The applicant states that no emergency protective measures are needed for low-water events. Other than a major breach of its embankment, a drop in water surface elevation in the main cooling reservoir will be gradual. The only safety-related water reservoirs proposed for STP Units 3 and 4 are the two engineered, partially buried UHS water-storage tanks (FSAR Figures 2.5S.4-49A through 2.5S.4-49D). The two UHS water-storage tanks, one for each proposed unit, will be located south of the respective units. The capacity of these UHS water-storage tanks will be sufficient to meet 30 days of cooling requirements under design-basis accident conditions, without needing any makeup or blowdown.

NRC Staff's Technical Evaluation

The staff issued RAI 02.04.14-1 requesting the applicant to describe severe hydrology-related events (levee breach, heavy rain, hurricane, tsunami, etc.) and to provide a summary of maximum water levels and available lead times to initiate and complete emergency procedures for each event in preparation for the main cooling reservoir EOPs in the future.

The applicant responded to RAI 02.04.14-1 in a letter dated January 28, 2009 (ML090300648). The applicant's response provides a list of events with the associated maximum water surface elevations and corresponding lead times at the STP Units 3 and 4 site. Table 2.4S.14-1 below summarizes this information.

Table 2.4S.14-1. Hydrological Events that Produce High Water Surface Elevations at STP Units 3 and 4 Site and Corresponding Lead Times

Hydrological Event	Water Surface Elevation (m / ft MSL)	Lead Time for Action	Basis for Determination of Lead Time
Postulated main cooling reservoir embankment breach	12.2 / 40	Greater than 30 minutes	Observation of main cooling reservoir conditions
Local intense precipitation	11.2 / 36.6	Greater than 2 hours	Flash flood or storm warnings from the National Weather Service
Multiple concurrent upstream dam failures	10.5 / 34.4	Between 58 and 65 hours	Notification from the Lower Colorado River Authority
Probable maximum flood in the Colorado River Basin	8.0 / 26.3	Flood does not reach site grade	Notification from the Lower Colorado River Authority
Probable maximum tsunami	3.5 / 11.5	Flood does not reach site grade	Post-event notification
Probable maximum hurricane	9.5 / 31.1	Flood does not reach site grade	Real-time monitoring by the National Hurricane Center
m=meter; ft=foot; MSL=mean sea level			

The applicant states that with the exception of the flood resulting from the main cooling reservoir embankment failure, sufficient time will be available to carry out site preparation activities such as ensuring an adequate supply of fuel oil, reducing floor drain sump inventories, ensuring the availability of sufficient maintenance personnel, ensuring the operation of emergency communication systems, sandbagging non-watertight entrances to buildings that are not safety-related, restoring watertight seals, and reducing low-level liquid waste inventories.

The applicant further states that emergency procedures for the main cooling reservoir breach will require closing watertight doors that are normally open and providing access to the control building. The applicant states that this is typically the only action necessary to ensure that safety-related equipment is safe from severe hydrology-related events.

The staff reviewed the applicant's information and determined that it is sufficient for future preparations of EOPs related to severe hydrology events. Therefore, RAI 02.04.14-1 is considered closed.

2.4S.14.5 Post Combined License Activities

The applicant identifies the following commitment:

- Commitment (COM 2.4S-1) – Develop EOPs for the main cooling reservoir that are similar to those provided for STP Units 1 and 2, before fuel loading.

2.4S.14.6 Conclusion

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to technical specification and emergency operations requirements. Based on the applicant's information, the staff determined that the main cooling reservoir embankment breach is the only severe hydrology-related event that may require EOPs. Therefore, no outstanding information is required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information relative to the TS and EOPs important to the operation of this plant. The staff accepted the methodologies used to determine the TS and emergency operations, as documented in SERs for previous licensing actions. Therefore, the staff found that the information addressing COL License Information Item 2.22 is adequate and acceptable. The staff concluded that the identified TS and emergency operations meet the requirements of 10 CFR 50.36, 10 CFR 52.79, 10 CFR 100.23(d), and 10 CFR 100.20(c).

2.4S.14.7 References

South Texas Nuclear Operating Company, "South Texas Project Combined License Application," Revision 0, Part 2, Final Safety Analysis Report, 2007.