# 2.0 SITE CHARACTERISTICS

This chapter of the application describes the site-specific characteristics that could affect the safe design and siting of the plant.

## 2.1S Geography and Demography

### 2.1S.1 Site Location and Description

#### *2.1S.1.1 Introduction*

This section of the Final Safety Analysis Report (FSAR) addresses the site boundaries and location of the site with respect to prominent natural and manmade features. This information demonstrates that the applicant has accurately described and appropriately used site characteristics in the plant design and operating criteria.

#### *2.1S.1.2 Summary of Application*

Section 2.1, "Limits Imposed on SRP Section II Acceptance Criteria by ABWR Standard Plant," of the South Texas Project (STP), Units 3 and 4, combined license (COL) FSAR Revision 12, incorporates by reference Section 2.1 of the certified Advanced Boling-Water Reactor (ABWR) design control document (DCD) Revision 4, referenced in Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," Appendix A, "Design Certification Rule for the U.S. Advanced Boiling Water Reactor," with no departures. In addition, in FSAR Section 2.1S.1, the applicant provides site-specific information on site location and a description that addresses COL License Information Item 2.3 as summarized below:

*COL License Information Item*

- COL License Information Item 2.3 Site Location and Description

COL License Information Item 2.3 addresses the provision of site-specific information including political subdivisions, natural and manmade features, population, highways, railways, waterways, and other significant features of the area.

This site-specific supplement included in the FSAR describes the following:

- Specification of State, county, and political subdivisions, in which the site is located, and location of the site with respect to natural and manmade prominent features (i.e., rivers, lakes; industrial, military, and transportation facilities); and
- Universal Transverse Mercator (UTM) co-ordinates (zone number, northing, and easting), meters; and latitude and longitude.
- Site Area Map consisting of the following:
  - Plant property lines, stating the area of plant property (in square kilometers [km2] [acres]);

– Location of site boundary, and location and orientation of principal plant structures within the site area (e.g., reactor building, auxiliary building, and turbine building);

– Location of any industrial, military, or transportation facilities and commercial, institutional, recreational, or residential structures within site area;

– Exclusion area distance (meters/feet) in all 16 cardinal compass directions;

– Scale that permits measurement of distances;

– True north; and

– Prominent natural and manmade features in the site area.

#### *2.1S.1.3 Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503, “Final Safety Evaluation Report Related to the Certification of the Advanced Boiling-Water Reactor Design,” Final Safety Evaluation Report ((FSER) related to the ABWR DCD).

In addition, the relevant requirements of the Commission regulations for the site location and description, and the associated acceptance criteria, are in Section 2.1.1, “Site Location and Description,” of NUREG–0800, “Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, (LWR Edition),” the Standard Review Plan (SRP). The regulatory basis for reviewing COL License Information Item 2.3 is in Section 2.1.1 of NUREG–0800.

The staff considered the following regulatory requirements in reviewing the applicant’s discussion of site location and description:

1. 10 CFR Part 50, “Domestic Licensing Of Production And Utilization Facilities,” and 10 CFR Part 52, as applicable in the FSAR of a detailed description and safety assessment of the site on which the facility is to be located, with appropriate attention to features affecting facility design (10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1)).

2. 10 CFR Part 100, “Reactor Site Criteria,” as it relates to the following: (1) defining an exclusion area and setting forth requirements regarding activities in that area (10 CFR 100.3), (2) addressing and evaluating factors that are used in determining the acceptability of the site as identified in 10 CFR 100.2(b), (3) determining an exclusion area such that certain dose limits would not be exceeded in the event of a postulated fission product release as identified in 10 CFR 50.34(a)(1) as it relates to site evaluation factors identified in 10 CFR Part 100, and (4) requiring that the site location and the engineered features included as safeguards against the hazardous consequences of an accident, should one occur, should ensure a low risk of public exposure.

Additional Regulatory Requirements include:

1. Specification of Location: The information submitted by the applicant is adequate and meets the requirements in 10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1), if it describes highways, railroads, and waterways that traverse the exclusion area in sufficient detail to allow the reviewer to determine that the applicant has met the requirements in 10 CFR 100.3, "Definitions."

2. Site Area Map: The information submitted by the applicant adequate and meets the requirements in 10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1) if it describes the site location, including the exclusion area and the location of the plant within the area, in sufficient detail to enable the reviewer to evaluate the applicant's analysis of a postulated fission product release, thereby allowing the reviewer to determine (in SRP Sections 2.1.2, "Exclusion Area Authority and Control," and 2.1.3, "Population Distribution," and in SRP Chapter 15, "Transient and Accident Analyses") that the applicant has met the requirements in 10 CFR 50.34(a)(1) and 10 CFR 100.3.

### *2.1S.1.4 Technical Evaluation*

As documented in NUREG–1503, the staff reviewed and approved Section 2.1 of the certified ABWR DCD. The staff reviewed Section 2.1S.1 of the STP, Units 3 and 4, COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to geography and demography.

The staff reviewed the following information in the COL FSAR:

*COL License Information Item*

- COL License Information Item 2.3 Site Location and Description

Specific information provided by the applicant to address COL License Information Item 2.3 includes:

- o The site layout and boundary for the proposed STP, Units 3 and 4, to be built on the site with respect to the existing STP, Units 1 and 2.
- o The site location with respect to political subdivisions and prominent natural and manmade features of the area within the 8.0-km (5-mile [mi]) low population zone (LPZ), 4.8-km (3-mi) LPZ and 80.5-km (50-mile) population zones

The staff independently estimated and verified the latitude and longitude and UTM coordinates of the proposed STP, Units 3 and 4 as provided in the FSAR.

Based on the staff's review of the information addressed in the STP, Units 3 and 4, FSAR, and also the staff's confirmatory review of pertinent information generally available in literature and

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

information collected during a site visit, the staff found that the applicant has provided information with regard to the site location that is adequate and acceptable.

The applicant provided the following information regarding the site area description:

- The topography and characteristics of the land surrounding the site for the proposed units.

- The commercial, industrial, institutional, recreational, and residential structures located within the site area.

- The distance from the proposed units to the nearest exclusion area boundary (EAB), including direction and distance.

- The distance of proposed STP, Units 3 and 4, to be built from regional, Federal, and State highways, railroads, and waterways that traverse or lie adjacent to the site. There are no recreational areas located within the STP site.

Except for STP, Units 1 and 2, no commercial, industrial, institutional, recreational, or residential structures are located within the STP site area.

No highways, railroads, or waterways traverse the exclusion area. No commercial, industrial, institutional, recreational, or residential structures are located within the STP site area. Therefore, there is no likelihood of any interference with normal plant operations from these sources.

Based on the staff's review of the site area information addressed in the FSAR, observations of the surrounding area of the STP site, and a review of the general information collected from the local officials during the site visit, the applicant's information with regard to the site location and area description is considered adequate and acceptable to allow the staff to evaluate whether the applicant meets the relevant requirements of 10 CFR 52.79 (a) (1) and 10 CFR Part 100 and satisfies the acceptance criteria specified in Section 2.1.1 of NUREG–0800. The staff verified that the EAB distance is consistent with the distance the applicant has used in the radiological consequence analyses described in Chapter 15 and in Section 13.3, "Conduct of Operations," of the FSAR.

The staff reviewed the applicant's proposal using the review procedures described in Section 2.1.1 of NUREG–0800.

#### *2.1S.1.5 Post Combined License Activities*

There are no post COL activities related to this section.

#### *2.1S.1.6 Conclusion*

The staff's finding related to information incorporated by reference is in NUREG–1503. The staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the site location and description that were incorporated by reference have been resolved.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations and the guidance in Section 2.1.1 of NUREG–0800. The staff's review concluded that the applicant has adequately addressed the COL license information item in accordance with Section 2.1.1 of NUREG–0800, which can be considered closed.

The staff finds that the applicant has provided sufficient information to support the determination of the acceptability of the site.

## 2.1S.2 Exclusion Area Authority and Control

### *2.1S.2.1 Introduction*

This section of the FSAR addresses the exclusion area authority and control. The applicant's legal authority to determine and control activities within the designated exclusion area is described. This authority establishes that the applicant has the authority to determine all activities, including exclusion and removal or personnel and property from the area.
This section also describes mineral rights and easements within the area.

### *2.1S.2.2 Summary of Application*

In Section 2.1S.2 of the STP Units, 3 and 4, COL FSAR Revision 12, the applicant provides site-specific information on exclusion area authority and control to address COL License Information Item 2.4 as summarized below.

*COL License Information Item*

- COL License Information Item 2.4 Exclusion Area Authority and Control

COL License Information Item 2.4 addresses the provision of site-specific information related to activities that may be permitted within the designated exclusion area.

The site-specific supplement included in the FSAR describes the following:

- Establishment of authority, which determines the legal authority of the land, mineral rights, and easements.
- Legal authority over all activities, including exclusion and removal of personnel and property from the area.
- Minimum distance and direction of EABs for present and proposed ownership.
- Activities unrelated to plant operation that are permitted in the EAB—their location, the nature of the activities, number of people involved, and plans for evacuation in the event of an emergency.
- Traffic control arrangements on highways, railroads and waterways traversing through the EAB in the event of an emergency.

- Procedures for abandonment, relocation, and understandings with other authorities for control.

### *2.1S.2.3 Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the exclusion area authority and control, and the associated acceptance criteria, are in Section 2.1.2 of NUREG-0800. The regulatory basis for reviewing COL License Information Item 2.4 is in Section 2.1.2 of NUREG–0800.

The staff considered the following regulatory requirements in reviewing the applicant's discussion of exclusion area authority and control:

1. 10 CFR Part 50 and 10 CFR Part 52, as they relate to the inclusion in the FSAR of a detailed description and safety assessment of the site on which the facility is to be located, with appropriate attention to features affecting facility design (10 CFR 50.34(a)(1), and 10 CFR 52.79(a)(1)).

2. 10 CFR Part 100, as it relates to the following: (1) defining an exclusion area and setting forth requirements regarding activities in that area (10 CFR 100.21(a), 10 CFR 100.3), (2) addressing and evaluating factors that are used in determining the acceptability of the site as identified in 10 CFR 100.20(b), and (3) determining an exclusion area such that certain dose limits would not be exceeded in the event of a postulated fission product release as identified in 10 CFR 50.34(a)(1), as it relates to site evaluation factors identified in 10 CFR Part 100.

Specific regulatory requirements include:

1. <u>Establishment of Authority</u>: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.33, "Contents of applications; general information," 10 CFR 50.34(a)(1), "Design objectives for equipment to control releases of radioactive material in effluents—nuclear power reactors," 10 CFR 52.79, "Contents of applications; technical information in final safety analysis report," and 10 CFR Part 100, if it provides sufficient detail to enable the staff to evaluate the applicant's legal authority within the designated exclusion area.

2. <u>Exclusion or Removal of Personnel and Property</u>: The information submitted by <u>the</u> applicant is adequate and meets the requirements of 10 CFR 50.33, 10 CFR 50.34(a)(1), 10 CFR 52.79, and 10 CFR Part 100, if it provides sufficient detail to enable the staff to evaluate the applicant's legal authority for the exclusion or removal of personnel or property from the exclusion area.

3. <u>Proposed and Permitted Activities</u>: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.33, 10 CFR 50.34(a)(1), 10 CFR 52.79, and 10 CFR Part 100, if it provides sufficient detail to enable the staff to evaluate the applicant's legal authority over all activities within the designated exclusion area.

### *2.1S.2.4 Technical Evaluation*

As documented in NUREG–1503, the staff reviewed and approved Section 2.1 of the certified ABWR DCD. The staff reviewed Section 2.1S.2 of the STP, Units 3 and 4, COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to exclusion area authority and control.

The staff reviewed the following information in the COL FSAR:

*COL License Information Item*

- COL License Information Item 2.4 Exclusion Area Authority and Control

The staff's review of COL License Information Item 2.4 follows. Specific information provided by the applicant to address the COL license information item includes:

  - Complete legal authority to regulate access and activity within the entire plant exclusion area.
  - Identification of facilities within the EAB that have authorized activities unrelated to plant operation, and emergency planning.
  - Arrangements for traffic control.
  - Abandonment or relocation of roads.

The STP participants (NRG Energy, CPS, City of Austin) own the land, including the mineral rights within the site boundary except for the rights of way for the public roads (Farm-to-Market Road [FM] 521, County Road 392 extending from FM 521 and adjacent to the western boundary of the site, and County Road 360, branching off the northeast corner of FM 521 as it loops around the site for meteorological tower access). The site boundary encompasses the designated EAB for STP, Units 3 and 4. STP participants have delegated to the STP Nuclear Operating Company (STPNOC) the authority to determine all activities within the EAB, including the exclusion and removal of personnel and property. STPNOC has authority over the EAB in the event of an emergency for the protection of public health and safety.

The staff verified the applicant's description of the exclusion area and the authority under which all activities within the exclusion area can be controlled. The staff also verified for consistency the EAB that is being considered for the radiological consequences in Chapter 15 and Section 13.3 of the FSAR by the applicant. The staff concluded that the applicant has the required authority to control all activities within the designated exclusion area.

No person or entity is allowed to reside, build, or conduct other activities within the designated EAB for STP, Units 3 and 4, without STPNOC's approval. The applicant stated that the facilities within the EAB in which authorized activities occur are the Visitor Center, which is located inside

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

the Nuclear Training Facility, and the Nuclear Training Facility itself. The Nuclear Training Facility is located inside the owner-controlled area and the EAB, but outside of the guard posts. All non-essential individuals in the EAB, including those in the Visitor Center, will be evacuated consistent with emergency planning procedures in the event of an emergency.

The staff verified that the emergency procedures for the EAB are addressed in Chapter 13.3 of this safety evaluation report (SER).

No Federal, State, or county roads or railways traverse the STP EAB. Therefore, there is no need for arrangements for traffic control.

Since there are no public roads within the STP, Units 3 and 4, EAB, there is no need to consider abandonment or relocation of roads. The staff reviewed the applicant's proposal using the review procedures described in Section 2.1.2 of NUREG–0800.

### *2.1S.2.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.1S.2.6 Conclusion*

The staff's finding related to information incorporated by reference is in NUREG–1503. The staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the exclusion area authority and control that were incorporated by reference have been resolved.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations and the guidance in Section 2.1.2 of NUREG–0800. The staff's review concluded that the applicant has adequately addressed the COL license information item in accordance with Section 2.1.2 of NUREG–0800.

The staff finds that the applicant has provided sufficient information for satisfying 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100.

## 2.1S.3 Population Distribution

### *2.1S.3.1 Introduction*

This section of the FSAR addresses the population distribution in the site vicinity. The review covers the following specific areas: population data, exclusion area, LPZ, nearest population center boundary, and population density.

### *2.1S.3.2 Summary of Application*

In Section 2.1S.3 of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant provides site-specific information on population distribution to address COL License Information Item 2.5 as summarized below.

*COL License Information Item*

- COL License Information Item 2.5 Population Distribution

This site-specific supplement included in the FSAR describes the following:

- Population data in the site vicinity, including transient populations.
- Population projections in the year of plant approval and 5 years thereafter.
- Population data consisting of information that includes the following:
  - Maps showing concentric circles with distances 1.6, 3.2, 4.8, 6.4, 8.0, and 16.1 km (1, 2, 3, 4, 5, and 10 mi) from the center of reactor units having background identifying cities, towns, and counties within around 16.1 km (10 mi); the circles are divided into 16 cardinal directions (e.g., true north through north-northwest).
  - A table providing current resident population with each area of the map formed by concentric circles and radial distances within 16.1 km (10 mi).
  - Projected population within 16.1 km (10 mi) in similar tabular form for the first year of plant operation.
  - Decennial projected population within 16.1 km (10 mi) through plant life in similar tabular form-description of the basis of and methodology for population projections and population data sources, including projections.

Tables and maps of suitable scale will depict the population distribution, including projections at 16.1-km (10-mi) intervals between 16.1- and 80.5-km (10- and 50-mi) radii from the center of the units for the first year of operation through plant life on the same decennial basis.

Also included are:

- Descriptions of seasonal variations in population due to activities, such as recreational and industrial activities, and inclusion of this population in current and projected population determinations.
- Evacuation plans for any residents.
- Evacuation plans in case of a potential accident.
- Nearest population center boundary (having 25,000 or more residents) is at least one and one-third times the distance from the reactor units to the outer boundary of the LPZ.
- Population density within 32.2 km (20 mi) is less than 193 people per square kilometer (500 people per square mile) to be consistent with the guidelines in Regulatory Position C.4 of Regulatory Guide (RG) 4.7, Revision 2, “General Site Suitability Criteria for Nuclear Power Stations.”

This site-specific supplement addresses COL License Information Item 2.5 of the ABWR DCD. COL License Information Item 2.5 from the certified ABWR DCD addresses the provision of population data for the site environs.

#### *2.1S.3.3 Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the population distribution, and the associated acceptance criteria, are in Section 2.1.3 of NUREG–0800. In particular, the relevant regulatory requirements are in 10 CFR Part 100.

The staff considered the following regulatory requirements in reviewing the applicant's discussion of site location and description:

1. 10 CFR 50.34(a)(1), as it relates to consideration of the site evaluation factors identified in 10 CFR 100.3, 10 CFR Part 100 (including consideration of population density), 10 CFR 52.79, as they relate to provisions from the applicant in the FSAR of the existing and projected future population profile of the area surrounding the site.

2. 10 CFR 100.20 and 10 CFR 100.21, as they relate to determining the acceptability of a site for a power reactor. In 10 CFR 100.3, 10 CFR 100.20(a), and 10 CFR 100.21(b), the NRC provides definitions and other requirements for determining an exclusion area, an LPZ, and population center distances

Specific acceptance criteria include:

1. <u>Population Data</u>: The population data supplied by the applicant in the FSAR is acceptable under the following conditions: (1) the FSAR contains population data from the latest census and projected populations in the year of plant approval and five years thereafter, in the geographical format in Section 2.1.3 of RG 1.70, Revision 3, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)," and in accordance with RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)," (2) the Safety Analysis Report (SAR) describes the methodology and sources used to obtain the population data, including the projections; (3) the SAR includes information on transient populations in the site vicinity.

2. <u>Exclusion Area</u>: The exclusion area should either not contains any residents or residents should be subject to immediate evacuation, if necessary as cited in NUREG–0800, Section 2.1.3.

3. <u>Low-Population Zone</u>: The specified LPZ is acceptable if it is determined that appropriate protective measures could be taken on behalf of the enclosed populace in the event of a serious accident as cited in NUREG–0800, Section 2.1.3.

4. <u>Nearest Population Center Boundary</u>: The nearest boundary of the closest population center containing 25,000 or more residents is at least one and one-

third times the distance from the reactor to the outer boundary of the LPZ as cited in NUREG–0800, Section 2.1.3.

5. Population Density: If the population density exceeds the guidelines given in Regulatory Position C.4 of RG 4.7, the applicant must give special attention to the consideration of alternative sites with lower population densities.

#### *2.1S.3.4 Technical Evaluation*

As documented in NUREG–1503, the staff reviewed and approved Section 2.1 of the certified ABWR DCD. The staff reviewed Section 2.1S.3 of the STP, Units 3 and 4, COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to population distribution.

The staff reviewed the following information in the COL FSAR:

*COL License Information Item*

- COL License Information Item 2.5 Population Distribution

The staff reviewed the specific information provided by the applicant to address COL License Information Item 2.5.

The staff noted that there are no residents in the exclusion area. The staff also reviewed the projected population data provided by the applicant. The population projections have been verified for consistency with the population projections addressed in Section 13.3 of this SER as part of emergency planning and preparedness. The staff also made confirmatory population projection estimates. The staff found the applicant's methodology for estimating population projections appropriate, reasonable, and acceptable.

Due to uncertainty in estimating the transient population between 16.1 and 80.5 km (10 and 50 mi) of the site and also due to the relatively small size of the expected transient population, the transient population is assumed to be insignificant compared with the residential population within an 80.5-km (50-mile) radius. Therefore, no transient population is considered between 16.1 and 80.5 km (10 and 50 mi) from the site. The staff found the applicant's estimate reasonable and acceptable.

The staff reviewed and confirmed the following information supplied by the applicant related to the LPZ for STP, Units 3 and 4:

- The LPZ for STP, Units 3 and 4, is the same as the LPZ for STP, Units 1 and 2, and consists of the area within a 4.8-km (3-mi) radius of a point 93 meters (m) (305 feet [ft]) directly west of the center of the STP, Unit 2 containment.

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification

- No towns, facilities, or institutions requiring special considerations for emergency planning purposes such as schools, nursing homes, hospitals, prisons, or major employers (other than STP) are known to exist within the LPZ or out to a distance of 8 km (5 mi).

- No transient or seasonal populations were identified in the LPZ. STP, Units 3 and 4, FSAR Figure 2.1S26, “Low Population Zone,” shows topographical features of the LPZ.

- The total population within the LPZ for the years 2000, through 2080, can be seen in FSAR Figures 2.1S-7, “Ten Miles 2000 Population Distribution,” through 2.1S-15, “Ten Mile 2080 Population Distribution”.

- The applicant evaluated representative design-basis accidents (DBAs) in Chapter 15 of STP, Units 3 and 4, COL FSAR, as discussed in Chapter 15 of this SER, to demonstrate that the radiological consequences of DBAs at the proposed site are within the dose limits in 10 CFR 50.34 (a)(1) as required by 10 CFR 100.21(c).

The staff verified that the closest population center having a population greater than 25,000 is Bay City, Texas, located approximately 19.2 km (12 mi) north-northeast of the STP site (well in excess of one and one-third times the distance of 4.8 km (3 mi) from the reactor to the outer boundary of the LPZ [10 CFR 100.21(b)]). Therefore, the staff concluded that the proposed site meets the population center distance requirement as defined in 10 CFR Part 100, Subpart B, “Evaluation Factors for Stationary Power Reactor Site Applications On or After January 10, 1997.”

Based on the staff’s verification of the applicant’s projected population data and population densities, assuming initial plant approval in the year 2015, and the start of plant operation in 2020, the staff found that the population density is well below the population density criterion of 193 persons per $km^2$ (500 persons per $mi^2$) averaged out to 32.2 km (20 mi) from the STP site. Therefore, the staff found that the application is consistent with Regulatory Position C.4 of RG 4.7, Revision 2.

### *2.1S.3.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.1S.3.6 Conclusion*

The staff’s finding related to information incorporated by reference is in NUREG–1503. The staff reviewed the application and checked the referenced DCD. The staff’s review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the population distribution that were incorporated by reference have been resolved.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations and the guidance in Section 2.1.3 of NUREG–0800. The staff’s review concluded that the applicant has adequately addressed the COL license information item in accordance with Section 2.1.3 of NUREG–0800.

The staff finds that the applicant has addressed the relevant information for satisfying 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100.

## 2.2S Nearby Industrial, Transportation, and Military Facilities

### 2.2S.1 Locations and Routes and Descriptions (Related to RG 1.206 Section C.I.2.2.1, "Locations and Routes," and Section C.I.2.2.2, "Descriptions")

#### *2.2S.1.1 Introduction*

Sections 2.2S.1, "Locations and Routes," and 2.2S.2, "Descriptions," of the FSAR, address the identification of potential hazards in the site vicinity. These sections describe potential external hazards or hazardous materials that are present or may reasonably be expected to be present during the projected lifetime of the proposed plant.

#### *2.2S.1.2 Summary of Application*

In Sections 2.2S.1 and 2.2S.2 of the COL FSAR Revision 12, the applicant provides site-specific information on locations and routes to address COL License Information Item 2.6 as summarized below.

*COL License Information Item*

- COL License Information Item 2.6 Identification of Potential Hazards in Site Vicinity

COL License Information Item 2.6, addresses the provision for information about industrial, military, and transportation facilities and routes to establish the presence and magnitude of potential external hazards.

The applicant identifies and addresses the potential hazard facilities and routes within the vicinity (8 km [5 mi]) of STP, Units 3 and 4, and airports within 16.1 km (10 mi) of the STP, along with other significant facilities beyond 8 km (5 mi), in accordance with RG 1.206 and relevant sections of 10 CFR Parts 50 and 100.

This site-specific supplement included in the FSAR addresses information that describes the following:

- Maps showing the location and distances from the nuclear units of all significant manufacturing plants, chemical plants, storage facilities, transportation routes (air, land, and water), transportation facilities, oil and gas pipelines, drilling operations, and extraction wells.
- Maps showing the facilities handling toxic, flammable, and explosive substances; nearby aircraft flight, holding, and landing patterns that may have the potential for adverse effects.
- A concise description of:
    - information on each facility including its primary function, major products, and the number of persons employed;

- the products and materials regularly handled, stored, used, or transported in the vicinity of the plant or on site;

- hazardous materials, including toxicity limits;

- statistical data on the amounts involved; modes of transportation; frequency of shipment; and the maximum quantity of hazardous materials likely to be processed, stored, or transported;

- pipelines; indication of pipe size, age, operating pressure, depth of burial, location, and type of isolation valves and type of gas or liquid being transported;

- navigable waterway information, including location of intake structures in relation to the shipping channel; the depth of the channel; the location of locks; the types of ships or barges using the waterway; and any nearby docks and anchorages;

- major highways and/or other roadways including the types of hazardous materials, the frequency of the transports, and the quantities being transported by truck in the vicinity of the STP site;

- identification of nearby railroads and information on the frequency and quantities of hazardous materials transported in the vicinity of the site;

- information on the length and orientation of runways, types of aircraft using the facility, number of operations per year by aircraft type, and the flying patterns associated with the airport;

- all airports within 8 km (5 mi) of the site;

- airports with projected operations greater than 500 x $d^2$ (where “d” is distance in miles from the site) movements per year within 16.1 km (10 mi) of the plant; and

- airports with projected operations greater than 1,000 x $d^2$ (where “d” is distance in miles from the site) movements per year beyond 16.1 km (10 mi) from the plant.

Equivalent information is included for aviation routes, pilot training areas, and landing and approach paths to airports and military facilities.

### *2.2S.1.3 Regulatory Basis*

The regulatory requirements of the Commission regulations for the nearby industrial, transportation, and military facilities, and the associated acceptance criteria, are in Section 2.2.1-2.2.2 of NUREG–0800. In particular the regulatory requirements are 10 CFR 100.20(b) and 10 CFR 52.79(a)(1)(iv).

Also, the acceptance criteria for identifying potential hazards in the site vicinity are based on meeting the relevant requirements in 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100.

The staff considered the following regulatory requirements in reviewing the applicant's discussion of site location and description:

1. 10 CFR 100.20(b), which requires that the nature and proximity of human-related hazards (e.g., airports, dams, transportation routes, military and chemical facilities) be evaluated to establish site parameters for use in determining whether plant design can accommodate commonly occurring hazards, and whether the risk of other hazards is very low.

2. 10 CFR 52.79(a)(1)(iv), as it relates to the factors to be considered in the evaluation of sites that require the location and description of industrial, military, or transportation facilities and routes, and of 10 CFR 52.79(a)(1)(vi) as it relates to compliance with 10 CFR Part 100.

Specific acceptance criteria include:

1. Data in the FSAR adequately describe the locations of and distances from the plant of nearby industrial, military, and transportation facilities and that such data are in agreement with data obtained from other sources, when available.

2. Descriptions of the nature and extent of activities conducted at the site and in its vicinity, including the products and materials likely to be processed, stored, used, or transported, are adequate to permit identification of the possible hazards cited in Subsection III of Section 2.2.1-2.2.2, "Identification of Potential Hazards in Site Vicinity," of NUREG–0800.

3. Sufficient statistical data with respect to hazardous materials are provided to establish a basis for evaluating the potential hazards to the plant or plants considered at the site.

### *2.2S.1.4 Technical Evaluation*

The staff reviewed the FSAR Sections 2.2S.1 and 2.2S.2, using the review procedures described in Section 2.2.1-2.2.2 of NUREG–0800.

*COL License Information Item*

- COL License Information Item 2.6 Identification of Potential Hazards in Site Vicinity

**Locations and Routes**

The applicant identified and provided information of potential external hazard facilities and operations within 8 km (5 mi) of STP, Units 3 and 4, which include five industrial facilities, five natural gas transmission pipelines, five chemical pipelines, four natural gas gathering pipelines, and five active natural gas and/or oil fields with active extraction wells. Major transportation routes within the vicinity of the site include four roads, two airways, and one navigable waterway.

The location of these facilities and road and waterway transportation routes are shown in STP, Units 3 and 4, FSAR Figure 2.2S-1, "Site Vicinity Map," and include:

- Industrial facilities within 8 km (5 mi) of the site:

  – OXEA Corporation (formerly known as Celanese)
  – Port of Bay City Operations
  – Gulfstream Terminal and Marketing
  – GulfMark Energy
  – STP, Units 1 and 2

- transportation routes within 8 km (5 mi):

  – FM 521
  – FM 1095
  – FM 1468
  – FM 3057
  – Colorado River (Waterway)

The location of natural gas and chemical pipelines and active natural gas and/or oil extraction fields are illustrated in STP, Units 3 and 4, FSAR Figure 2.2S-2, “Pipeline Oil/Gas Well Map,” and include:

- Natural Gas Transmission pipelines:

  – Dow Pipeline Company
  – Houston Pipeline Company, L.P.
  – Penn Virginia Oil & Gas, L.P.
  – Texas Eastern Transmission, L.P.
  – Enterprise Products Operating, L.P.

- Chemical Pipelines:

  – Seadrift Pipeline Corporation (ethylene gas)
  – OXEA Corporation (propylene)
  – OXEA Corporation (oxygen)
  – OXEA Corporation (nitrogen)
  – OXEA Corporation (ethylene)

- Natural Gas Gathering Pipelines:

  – Acock/Anaqua Operating Co., L.P.
  – Houston Pipeline Company, L.P.
  – Kinder Morgan Tejas Pipeline, L.P.
  – Santos USA Corporation

- Natural Gas/Oil Extraction Fields:

  – Duncan Slough
  – Cane Island
  – Petrucha
  – Grand Slam
  – Wadsworth

STP, Units 3 and 4, FSAR Figure 2.2S-1, illustrates industrial facilities and transportation routes within 16.1 km (10 mi) of the site and includes industrial facilities within 8.0 to 16.1 km (5 to 10 mi), Equistar Industries, and the Matagorda Waste Disposal and Water Supply Corporation (WWTP).

STP, Units 3 and 4, FSAR Figure 2.2S-3, “Airport/Airways Map,” illustrates airports and airway routes within 16.1 km (10 mi) of the site and includes airport and airway routes within 16.1 km (10 mi) of the site, the STP Corporate Helipad, Airway V-70, and Airway V-20.

**Descriptions**

The industrial facilities and transportation routes identified and addressed above are described as follows:

*Descriptions of Facilities*

The six facilities described include: STP, Units 1 and 2; OXEA Corporation; Gulfstream Terminal and Marketing, LLC.; GulfMark Energy; Equistar, and Matagorda Waste Disposal and Waste Water Supply Corporation. FSAR Table 2.2S-1, provides concise descriptions of and information about each facility.

*Descriptions of Products and Materials*

STP, Units 1 and 2, are located approximately 457 m (1,500 ft) southeast of STP, Units 3 and 4. There are approximately 1,300 people currently employed at STP, Units 1 and 2. The chemicals identified for possible analysis and their locations at STP, Units 1 and 2, are presented in FSAR Table 2.2S-2.

OXEA Corporation is a chemical manufacturing facility located approximately 6.9 km (4.3 mi) north-northeast of STP, Units 3 and 4. A variety of chemical products are produced at the site, including organic chemicals (basic and industrial), cyclic organic crudes, organic dyes, and pigments. OXEA employs 260 persons. FSAR Table 2.2S-3, summarizes the quantity of hazardous materials currently stored at the plant and the applicable toxicity limits.

OXEA Corporation receives and ships materials by rail, truck, barge, and pipeline. The facility ships tank rail cars on the Union Pacific rail line spur that travels from Bay City to Blessing. Tank rail cars are also shipped on the Burlington Northern Santa Fe rail line that runs east from the plant’s main line and then to Bay City. The tank trucks are shipped and received via FM 3057 and FM 2668. Neither the truck nor the rail transport routes approach closer to STP, Units 3 and 4, than the storage location of the chemicals at OXEA. OXEA Corporation also ships materials in barges along the Colorado River. Approximately 360 barges per year transit the Colorado River. There are four pipelines that carry products into the plant.

The Port of Bay City is a port facility located adjacent to OXEA Corporation along the Colorado River, approximately 7.4 km (4.6 mi) north-northeast of STP, Units 3 and 4. There are two facilities located at this Port: Gulfstream Terminal and Marketing, LLC. and GulfMark Energy.

Gulfstream Terminal and Marketing, LLC. receives barge shipments of refined petroleum products such as gasoline and diesel fuel and stores the products until they are delivered by truck to retail terminals. FSAR Table 2.2S-3, summarizes the maximum quantity of potentially hazardous materials stored at the terminal and the applicable toxicity limits. Gulfstream employs four workers.

GulfMark Energy is also located 7.4 km (4.6 mi) north-northeast of STP, Units 3 and 4, at Bay City. This terminal is used to receive, store, and transfer petroleum crude oil and condensate. The facility has an average monthly inventory of 1987.5 $m^3$ (12,500 barrels). The oil is offloaded in 28.6 $m^3$ (180-barrel [7,560 gallon]) truckloads. FSAR Table 2.2S-3, summarizes the maximum quantity of potentially hazardous materials stored at the terminal and the applicable toxicity limits.

Equistar Chemicals (Equistar), a subsidiary of Lyondell Chemical Company, is located 11.3 km (7 mi) east of STP, Units 3 and 4. Equistar employs 194 people and produces high-density polyethylene (HDPE) plastic resins. This facility receives and ships materials by both rail and truck. Truck transport is via State Highway 60 due to bridge limitations on FM 521. Chemicals are stored or situated at distances greater than 8 km (5 mi) from the plant.

Matagorda Waste Disposal and Water Supply Corporation are located approximately 14.5 km (9 mi) southeast of STP, Units 3 and 4. Matagorda employs three workers and receives chemicals for treatment by truck transport via State Highway 60. Chemicals are stored or situated beyond 8 km (5 mi) from the STP site.

*Descriptions of Pipelines and Natural Gas/Oil Fields*

There are five natural gas transmission pipelines, five chemical pipelines, four natural gas gathering pipelines, and five natural gas and/or oil fields with active extraction wells within 8 km (5 mi) of STP, Units 3 and 4, as depicted in FSAR Figure 2.2S-2. Information pertaining to these pipelines is also presented in FSAR Table 2.2S-4, “STP 3 & 4 Pipeline Information Summary.”

The natural gas transmission pipelines that may also serve the following gas and/or oil fields—Duncan Slough, Cane Island, Petrucha, Grand Slam, and Wadsworth—are described below:

Dow Chemical Company operates two natural gas transmission pipelines at the closest distance of 3.2 km (2 mi) northwest of STP, Units 3 and 4. Dow Collegeport has a 32.4-centimeter (cm) (12.75-inch [in.]) diameter pipeline with an operating pressure of 3.25 megapascals gauge (MPaG) (471 pounds per square inch gauge [psig]), and Dow Powderhorn has a 40.6-cm (16-in.) diameter pipeline with an operating pressure of 5.2 MPaG (760 psig). Both are buried at a depth of 0.9 to 3.0 m (3 to 10 ft).

The Houston Pipeline Company operates a natural gas transmission pipeline that passes within 4.5 km (2.8 mi) north of STP, Units 3 and 4. The pipeline is 21.9 cm (8.63 in.) in diameter with an operating pressure of 4.0 MPaG (575 psig). The pipeline is buried at a depth of 0.6 to 0.9 m (2 to 3 ft) with a distance of 11.3 to 12.9 km (7 to 8 mi) between isolation valves.

The Penn Virginia Oil & Gas operates a natural gas transmission pipeline 11.4 cm (4.5 in.) in diameter that passes within 6.1 km (3.8 mi) northeast of STP, Units 3 and 4.

Texas Eastern Transmission operates a 76.2-cm (30-in.) natural gas transmission pipeline that passes within 6.8 km (4.2 mi) north of STP, Units 3 and 4.

Enterprise Products Operating operates a 21.9-cm (8.63-in.) natural gas transmission pipeline that passes within 6.8 km (4.2 mi) north of STP, Units 3 and 4, with an operating pressure of 5.2 MPaG (750 psig). The pipeline is buried at an average depth of 94 cm (37 in.).

The chemical pipelines include the following:

- Seadrift Pipeline Company operates an 11.4-cm (4.5-in.) diameter nitrogen pipeline buried at a depth of 0.9 to 3.0 m (3 to 10 ft), with an operating pressure of 10.3 MPaG (1,494 psig), 5.6 km (3.5 mi) north of STP, Units 3 and 4.

- OXEA Corporation owns a 16.8-cm (6.63-in.) propylene line buried at a depth of 96.5 to 101.6 cm (38 to 40 in.), with an operating pressure of 6.0 MPaG (875 psig). The pipeline delivers products into the OXEA plant and passes within 6.9 km (4.3 mi) north-northeast of STP, Units 3 and 4.

- Air Liquide operates a 32.4-cm (12.75-in.) oxygen pipeline to the OXEA plant, buried at a depth of 96.5 to 101.6 cm (38 to 40 in.) with an operating pressure of 6.0 MPaG (875 psig). The pipeline passes within 6.9 km (4.3 mi) north-northeast of STP, Units 3 and 4. Air Liquide also operates another 27.3-cm (10.75-in.) nitrogen pipeline to the OXEA plant.

- Equistar operates a 27.3-cm (10.75-in.) ethylene pipeline to the OXEA plant buried at a depth of 1.2 to 1.8 m (4 to 6 ft), with an operating pressure of 6.9 to 9.0 MPaG (1,000 to 1,300 psig). The pipeline passes within 6.9 km (4.3 mi) north-northeast of STP, Units 3 and 4.

The natural gas gathering pipelines are described as follows:

- Acock/Anaqua Operating Company operates an 11.4-cm (4.5-in.) natural gas gathering pipeline serving the South Duncan Slough field that terminates 2.1 km (1.3 mi) northwest of STP, Units 3 and 4.

- The Houston Pipeline Company operates an 11.4-cm (4.5-in.) natural gas gathering pipeline serving the Duncan Slough field and passing within 5.3 km (3.3 mi) north of STP, Units 3 and 4.

- The Kinder Morgan Tejas Pipeline Company operates 40.6-cm (16-in.) natural gas gathering pipeline that passes within 7.1 km (4.4 mi) northwest of STP, Units 3 and 4.

- The Santos USA Corporation operates an 11.4-cm (4.5-in.) natural gas gathering pipeline that passes within 4.8 km (3 mile) north-northwest of STP, Units 3 and 4.

*Descriptions of Waterways*

The STP, Units 3 and 4, site is located approximately 5.1 km (3.2 mi) from the west bank of the Colorado River, a navigable waterway. From the Gulf Intracoastal Waterway, the river winds along a 25.1 km (15.6-mi) stretch until it approaches the turning basin located at the Port of Bay City facility, approximately 7.4 km (4.6 mi) north-northeast of STP, Units 3 and 4. The Port of Bay City is the only dock/anchorage located within 8 km (5 mi) of the STP site.

The Colorado River is used primarily for barge traffic. During 2005, there was a total of 208 barge and 314 tanker inbound trips, and 211 barge and 322 tanker outbound trips are recorded. These vessels primarily used the river for the transportation of raw and finished materials to local industrial facilities, predominantly OXEA Corporation and the Port of Bay City terminals. These commodities included 50,802 metric tons (56,000 short tons) of crude petroleum, 907 metric tons (1,000 short tons) of residual fuel oil, 115,212 metric tons (127,000 short tons) of alcohols, and 287,578 metric tons (317,000 short tons) of carboxylic acids. FSAR Table 2.2S-5, "Hazardous Chemical Waterway Freight, Colorado River," details the total quantity of hazardous materials transported on the Colorado River in the vicinity of STP, Units 3 and 4.

*Descriptions of Highways*

Matagorda County is traversed by several highways. There are four FMs within 8 km (5 mi) of STP, Units 3 and 4, as depicted in FSAR Figure 2.2S-1. FM 521 is the road with the closet approach to STP, Units 3 and 4. At its closest point, FM 521 is approximately 0.6 km (0.4 mi) from STP, Units 3 and 4, and runs in an east-west direction parallel to the STP site's northern fence. To the north of the STP site, FM 1468 runs in a north-south direction and intersects FM 521 approximately 1.6 km (1 mi) from STP, Units 3 and 4. FM 521 intersects FM 1095, which also runs in a north-south direction and is approximately 6.8 km (4.2 mi) to the west of STP, Units 3 and 4. Another road located in the vicinity of STP, Units 3 and 4, is FM 3057, which runs in an east-west direction and is located north-northeast of STP, Units 3 and 4. FM 3057 links OXEA Corporation with FM 2668. Each of the on-site chemicals that have the potential to explode or form a flammable or toxic vapor cloud was analyzed to determine a safe distance. FM 521 closest approach to the nearest safety-related structure is 595.9 m (1,955 ft), and to the nearest control room is 869.6 m (2,853 ft). The distance from the on-site chemical storage is closer compared to the distance from FM 521 to either the identified safety-related structure or control room.

*Descriptions of Railroads*

There are no railroads in the vicinity (8 km [5 mi]) of STP, Units 3 and 4.

*Descriptions of Airports*

Only one helipad, the STP helipad, is located within the vicinity (8 km [5 mi]) of STP, Units 3 and 4. An average of two to three corporate flights per year use the helipad.

There are no airports located within 8 km (5 mi) of the STP site. In addition, there are no airports within 16.1 km (10 mi) of the site with projected operations greater than 500 $d^2$ per year, or beyond 16.1 km (10 mi) with projected operations greater than 1,000 $d^2$ per year, where "d" is distance in statue miles from the site. The closest municipal airport is Palacios Municipal Airport with 3,000 operations per year. Although small, private airstrips may be present in this area, the flights are sporadic and do not pose a threat to the STP site.

The center line of Airway V-70 is approximately 5.6 km (3.5 mi) northwest of the STP site, and the center line of Airway V-20 is approximately 15.4 km (9.6 mi) northwest of the STP site, as depicted in FSAR Figure 2.2S-3. The width of a federal airway is 14.8 km (eight nautical miles), 6.4 km (4 mi) on each side of the center line, and this places the V-70 airway closer to the plant than 3.2 km (2 mi) to the nearest edge. Therefore, the probability of aircraft accidents due to operations along this Airway V-70 that could possibly result in radiological consequences for the STP site was estimated and met the NUREG–0800 criterion of about $10^{-7}$ per year.

*Projections of Industrial Growth*

Based on the Office of Economic Development and the staff contact from the Chamber of Commerce, it is assumed that there are no known major plans to develop any industrial facilities within 8 km (5 mi) of the STP site. However, there would be some growth potential expected due to the construction and operation of STP, Units 3 and 4.

#### *2.2S.1.5 Post Combined License Activities*

There are no post COL activities related to this subsection.

#### *2.2S.1.6 Conclusion*

The staff reviewed the information in Sections 2.2S.1 and 2.2S.2, of the COL FSAR against 10 CFR 100.20b and 10 CFR 52.79(a)(1)(iv) and found that the applicant has provided sufficient information with respect to the identification of potential hazards in the site vicinity.

The staff confirmed that the applicant has evaluated the nature and extent of activities involving potentially hazardous materials that are conducted at nearby industrial, military, and transportation facilities to identify any such activities that have the potential for adversely affecting plant safety-related structures.

The staff's review confirmed that the applicant has adequately addressed COL License Information Item 2.6 in accordance with Section 2.2.1-2.2.2 of NUREG–0800, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

Based on an evaluation of information in the COL FSAR as well as information that the staff independently obtained, the staff finds that all potentially hazardous activities on site and in the vicinity of the plant have been identified. The hazards associated with these activities have been reviewed and are discussed in Sections 2.2S.3, 3.5.1.5, and 3.5.1.6 of this SER.

### 2.2S.2 Descriptions

This section of the FSAR is evaluated in SER Section 2.2S.1.

### 2.2S.3 Evaluation of Potential Accidents

#### *2.2S.3.1 Introduction*

This section of the FSAR addresses the applicant's identification and evaluation of potential accident situations in the vicinity of the plant. The application includes probability analyses of potential accidents involving hazardous materials or activities on site and in the vicinity of the proposed site.

#### *2.2S.3.2 Summary of Application*

In Section 2.2S.3 of the COL FSAR Revision 12, the applicant provides a site-specific evaluation of information identified in COL FSAR Sections 2.2S.1 and 2.2S.2 for the potential accidents that should be considered as design-basis events, and potential effects of these accidents on the nuclear plant to address COL License Information Item 2.7 as summarized below.

*COL License Information Item*

- COL License Information Item 2.7 Evaluation of Potential Accidents

COL License Information Item 2.7, addresses the evaluation of potential accidents and their effects on the operation of STP, Units 3 and 4.

Section 2.2S.3 of the STP, Units 3 and 4, COL FSAR Revision 12 provides the following:

The applicant identifies and evaluates information regarding potential accidents considered as DBAs that may affect the STP, Units 3 and 4, in terms of design parameters (e.g., overpressure or missile energies) and physical phenomena (e.g., concentration of flammable or toxic vapor clouds outside of the building structures). DBAs internal and external to the nuclear plant are defined as those accidents that have a probability of occurrence on the order of magnitude of $10^{-7}$ per year or greater with potential consequences serious enough to affect the safety of the plant to the extent that the guidelines in 10 CFR Part 100 could be exceeded.

This site-specific supplement included in the FSAR describes the following:

- Evaluation of hazards associated with nearby industrial activities, such as manufacturing, processing, or storage facilities.
- Evaluation of hazards associated with nearby military activities, such as military bases, training areas, or aircraft flights.
- Evaluation of hazards associated with nearby transportation routes (airways, highways, railways, navigable waters, and pipelines).

The principal types of hazards considered for evaluation with respect to each of the above areas include the following:

- Toxic vapors or gases and their potential for incapacitating nuclear power plant control room operators.
- Overpressure resulting from explosions or detonations involving materials such as munitions, industrial explosives, or explosive vapor clouds resulting from the atmospheric release of gases with the potential for ignition and explosion.
- Missile effects attributable to mechanical impacts such as aircraft impacts, explosion debris, and impacts from waterborne items such as barges.
- Thermal effects attributable to fires.

Based on the information provided in FSAR Sections 2.2S.1 and 2.2S.2 pertaining to the identification of potential hazards, the applicant determines the potential accidents that are to be considered DBAs and identifies the potential effects on the plant from those accidents in terms of design parameters (e.g., overpressure, missile energies) or physical phenomena (e.g., the concentration of a flammable or toxic cloud outside of the building structures).

Accident categories for selecting design-basis events include explosions, flammable vapor clouds, toxic chemicals, fires, collisions with intake structures, and liquid spills and cover the following:

> Accidents involving detonations of high explosives, munitions, chemicals, or liquid and gaseous fuels for facilities and activities in the vicinity of the plant or on site, where materials are processed, stored, used, or transported in quantity are considered.
>
> Accidental releases of flammable liquids or vapors that result in the formation of unconfined vapor clouds are considered.
>
> Assuming no explosion occurs, the calculation of the extent of the cloud and concentration of gas that could reach the plant under the worst-case meteorological conditions is determined.
>
> The releases of toxic chemicals from on-site storage facilities and nearby mobile and stationary sources are evaluated under the worst meteorological conditions. These calculated chemical concentrations are considered in the evaluation of control room habitability in Section 6.4, "Habitability Systems," of the FSAR.
>
> Accidents leading to high heat fluxes or smoke and nonflammable gas or chemical release as the consequence of fires in the vicinity of the plant are evaluated. Evaluation of fires in adjacent industrial and chemical plants, storage facilities, oil and gas pipelines, brush and forest fires, and fires from transportation accidents that lead to high heat fluxes or the formation of clouds are evaluated under the worst meteorological conditions. These calculated concentrations are considered in the evaluation of control room habitability in Section 6.4 of the FSAR.
>
> For the navigable waterways, the evaluation considers the probability of and potential effects of impact on the plant cooling water intake structure and enclosed pumps from passing barges or ships, including any explosions incident to the collision.
>
> The release of oil or liquids due to spills could affect the plant's safe operation are considered.
>
> Particular attention is given to potential accidental explosions that could produce a blast overpressure of 6.9 kilopascals (kPa) (1 pound per square inch [psi]) or greater, using quantity-distance relationships.

This site-specific supplement addresses COL License Information Items 2.7, 2.8, and 2.42 from the ABWR DCD.

- COL License Information Item 2.7 Evaluation of Potential Accidents

This COL license information item identifies potential accident scenarios in the vicinity of the plant and the bases for which these potential accidents are or are not accommodated in the design.

- COL License Information Item 2.8 External Impact Hazards

This COL license information item addresses the review and evaluation of the effects on the protection criteria of some external impact hazards, such as general aviation or nearby explosions.

- COL License Information Item 2.42 CRAC 2 Computer Code Calculations

This COL license information item addresses the use of the CRAC-2 computer code to verify compliance with acceptance criteria, data input, and severe accident analyses for the determination of ABWR site acceptability for severe accidents. CRAC 2 computer code is replaced with MACCS2 computer code through Departure STD DEP 2.2-5, which in turn is evaluated in Chapter 19 of this SER.

### *2.2S.3.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the evaluation of potential accidents, and the associated acceptance criteria, are in Section 2.2.3, "Evaluation of Potential Accidents," of NUREG–0800 and RG 1.91, "Evaluations of Explosions Postulated To Occur on Transportation Routes Near Nuclear Power Plants." The regulatory requirements for reviewing the COL license information items is 10 CFR 52.79(a)(1)(iv).

In particular, the staff considered the following regulatory requirements in reviewing the applicant's discussion of potential accidents:

10 CFR 52.79(a)(1)(iv), as it relates to the factors to be considered in the evaluation of sites, which require the location and description of industrial, military, or transportation facilities and routes and the requirements in 10 CFR 52.79(a)(1)(vi) as they relate to compliance with 10 CFR Part 100.

Specific regulatory requirements include:

1. <u>Event Probability</u> The identification of design-basis events resulting from the presence of hazardous materials or activities in the vicinity of the plant or plants of a specified type is acceptable, if all postulated types of accidents are included for which the expected rate of occurrence of potential exposures resulting in radiological dose in excess of the 10 CFR 50.34(a)(1) limits, as it relates to the requirements of 10 CFR Part 100, is estimated to exceed the staff objective of an order of magnitude of $10^{-7}$ per year.

2. <u>Design-Basis Events</u> The effects of design-basis events have been adequately considered, in accordance with 10 CFR 100.20(b), if analyses of the effects of those accidents on the safety-related features of the plant or plants of a specified type have been performed and measures have been taken (e.g., hardening, fire protection) to mitigate the consequences of such events.

### *2.2S.3.4 Technical Evaluation*

The staff reviewed the FSAR Section 2.2S.3 using the review procedures described in Section 2.2.3 of NUREG–0800.

*COL License Information Items*

- COL License Information Item 2.7 Evaluation of Potential Accidents
- COL License Information Item 2.8 External Impact Hazards

**Determination of Design-Basis Events**

The applicant analyzed postulated accidents for various types and considered the identified sources and locations of accidents in FSAR Section 2.2S.1, which includes the following:

- Explosions
- Flammable Vapor Clouds
- Release of Hazardous Chemicals (Toxic Chemicals)
- Fires
- Collision with Intake Structures
- Liquid Spills
- Radiological Hazards

*Explosions*

The applicant considers the potential for explosions resulting in blast overpressures due to detonation of explosives, munitions, chemicals, liquid fuels, and gaseous fuels for facilities and activities either on site or within the site vicinity of the proposed plant. The blast overpressure of 6.9 kPa (1 psi) that could adversely affect the plant operation or would prevent the safe shutdown of the plant from explosions from nearby railways, highways, navigable waterways, or facilities to safety-related structures were evaluated by the applicant. The value of 6.9 kPa (1 psi) of peak positive incident overpressure was considered based on RG 1.91, Revision 1, "Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants," below which no significant damage would be expected.

Onsite chemicals, offsite chemicals, and hazardous materials transported on navigable waterways are addressed in the STP, Units 3 and 4, COL FSAR and in Tables 2.2S-6, "Onsite Chemical Storage – Disposition"; 2.2S-7, "Offsite Chemicals, Disposition - OXEA Corporation, Gulfstream Terminal and Marketing LLC, and GulfMark Energy"; and 2.2S-8, "Hazardous Materials, Navigable Waterway Transportation – Disposition"; respectively. The applicant evaluates hazardous materials potentially transported on FM 521 and natural gas transported in pipelines to ascertain which hazardous materials have the potential to explode. The applicant stated that the evaluations are in accordance with RG 1.91, Revision 1, conservative assumptions in NUREG–1805, "Fire Dynamics Tools (FTDs) Quantitative Fire Hazard Analysis Methods for the U.S. Nuclear Regulatory Commission Fire Protection Inspection Program," and FSAR Reference FM Global ("Guidelines for Evaluating the Effects of Vapor Cloud Explosions Using a TNT Equivalency Method," Factory Mutual Insurance Company, dated May 2005). The effects of these explosion events in terms of minimum safe distances from internal and external sources are summarized in STP, Units 3 and 4, FSAR Table 2.2S-9, "Design-Basis Events – Explosions." The staff conducted an independent analysis using RG 1.91, Revision 1, and the results were not comparable to the results submitted by the applicant. The staff issued a request for information (RAI) 02.02.03-1, asking for a more detailed explanation of the methodology the applicant used to perform the explosion analyses. In its response to RAI 02.02.03-1, dated May 29, 2008 (ML081560702), the applicant provides a detailed methodology that will be included as an appendix to the FSAR. The applicant's response points out that the use of RG 1.91 for atmospheric liquids is overly conservative, and the accompanying detailed methodology provides an alternative approach.

The staff reviewed the applicant's response to RAI 02.02.03-1, and found the approach to be generally reasonable. The staff performed confirmatory calculations using more conservative assumptions. With the more conservative assumptions, some of the results had greater minimum safe distances at which 6.9 kPa (1 psi) overpressure would not be exceeded than the applicant's calculated distances. Nevertheless, the staff's calculated distances were less than the corresponding minimum separation distance from the safety-related structure as indicated in RG 1.91, Revision 1. Therefore, the staff found that the chemicals, their quantities, and locations identified in the application pose no threat to the safe operation of the plant and confirmed the applicant's conclusion. Therefore, RAI 02.02.03-1 is resolved and closed.

*Flammable Vapor Clouds (Delayed Ignition)*

Flammable materials in the liquid or gaseous state can form an unconfined vapor cloud that can drift toward the plant before an ignition event. Flammable chemicals released into the atmosphere can form vapor clouds, dispersing as they travel downwind, and the portion of the cloud in between the lower flammable limit (LFL) and upper flammable limit (UFL) may burn if the cloud encounters an ignition source. This encounter may lead to an explosion.

The applicant considered the potential chemicals pertaining to on-site and offsite chemical storage; hazardous materials transported on navigable waterways (presented in STP, Units 3 and 4, FSAR Tables 2.2S-6, 2.2S-7, and 2.2S-8); and hazardous materials transported on FM 521. The applicant conducted an evaluation to ascertain which materials had the potential to form flammable vapor clouds and vapor cloud explosions. The applicant uses ALOHA and DEGADIS models in determining the distances for the vapor cloud to be present in the flammable range and the potential minimum distance not to exceed 6.9 kPa (1 psi) overpressure due to this vapor cloud explosion. The applicant presents the results of these analyses in STP, Units 3 and 4, FSAR, Table 2.2S-10, "Design-Basis Events, Flammable Vapor Clouds (Delayed Ignition) and Vapor Cloud Explosions." The applicant concluded that a flammable vapor cloud with the possibility of ignition or explosion from any of the above addressed facilities and transportation routes will not adversely affect the safe operation or shutdown of STP, Units 3 and 4.

To be able to perform independent confirmatory analyses for all of the chemicals/hazardous materials that the applicant addressed, the staff required further information regarding the inputs the applicant used in its modeling. In RAI 02.02.03-2 and follow-up RAI 02.02.03-3, the staff requested this additional information from the applicant. In its responses to RAI 02.02.03-2, dated May 29, 2008 (ML081560702), and to RAI 02.02.03-3, dated October 27, 2008 (ML083040527), the applicant allowed the staff to perform the analyses using the ALOHA model (ALOHA, 2007). The staff used conservative assumptions in formulating the scenario and also in the ALOHA model analyses. The meteorological inputs used in the ALOHA modeling included F (stable) stability class with a wind speed of 1 m (3,3 ft) per second (representing the worst 5 percent of meteorological conditions); an ambient temperature of 25 degrees Celsius (°C) (77 degrees Fahrenheit [°F]); relative humidity of 50 percent; and a cloud cover of 50 percent. For each of the identified chemicals in the liquid state, the staff conservatively assumed that the entire contents of the vessel leaked, forming a 1-cm-thick (0.4-in.-thick) puddle. This assumption provided a significant surface area from which to maximize the evaporation and formation of a vapor cloud. Since the ALOHA model is limited by the maximum surface area of 31,400 $m^2$ (7.76 acres), for those chemical inventories that gave a 1-cm (0.4-in.) puddle greater than this limiting surface area, the calculated evaporation rate based on the limiting surface area of 31,400 $m^2$ (7.76 acres) was adjusted to reflect the actual inventory of the chemical and was modeled further as a direct source option. For each of the identified chemicals in a gaseous state, it was conservatively assumed that the entire contents were released over a ten-minute period as a continuous direct source. The results of these analyses were comparable or sometimes higher than those of the applicant's results. However, the minimum distance calculated due to an explosion of a flammable chemical vapor cloud for the incident pressure of 6.9 kPa (1 psi) did not exceed the respective nearest distance to a safety-related structure. Therefore, the staff found that the potential explosion of a flammable vapor cloud from any of the facilities and transportation routes addressed would not have an adverse impact on the safe operation of STP, Units 3 and 4.

*Toxic Chemicals*

Accidents involving the release of toxic chemicals from on-site storage facilities and nearby mobile and stationary sources were considered. The applicant considers the potentially hazardous chemicals pertaining to on-site and offsite chemical storage; hazardous materials transported on navigable waterways (presented in STP, Units 3 and 4, FSAR Tables 2.2S-6, 2.2S-7, and 2.2S-8); and hazardous materials transported on FM 521. The applicant performed an evaluation to ascertain which materials had the potential to form a toxic vapor cloud following an accidental release. The applicant mainly uses the ALOHA model, and the toxic dispersion model only was used for barge transport of gasoline, to predict the concentrations of toxic chemical clouds as they disperse downwind for all facilities.
The maximum distance a cloud could travel before it disperses enough to fall below the immediate danger to life and health (IDLH) concentrations in the vapor cloud is determined using the ALOHA model. The ALOHA model is also used to predict the concentration of the chemical in the control room following a chemical release to ensure that, under the worst-case scenarios, control room operators would have sufficient time to take appropriate protective action. The applicant presents the results of these analyses in STP, Units 3 and 4, FSAR Table 2.2S-11, "Design-Basis Events, Toxic Vapor Clouds," and concludes that the formation of a toxic vapor cloud, following an accidental release from any of the above addressed facilities and transportation routes, will not adversely affect the safe operation or shutdown of STP, Units 3 and 4.

To be able to perform independent confirmatory analyses for the applicant's addressed chemicals/hazardous materials, the staff required further information regarding the inputs the applicant had used in the modeling. In RAIs 02.02.03-4 and 02.02.03-5, the staff requested additional information from the applicant. In its responses to RAI 02.02.03-4, dated November 20, 2008 (ML083290340), and to RAI 02.02.03-5, dated October 27, 2008 (ML083040527), the applicant allowed the staff to perform the analyses using the ALOHA model (ALOHA, 2007). The staff used conservative assumptions in formulating the scenario and also in the ALOHA model analyses. The meteorological inputs used in the ALOHA modeling included F(stable) stability class with a wind speed of 1 m (3.3 ft) per second (which represented the worst 5 percent of meteorological conditions); an ambient temperature of 25 °C (77 °F), a relative humidity of 50 percent; and a cloud cover of 50 percent. For each of the identified chemicals in the liquid state, it was conservatively assumed that the entire contents of the vessel leaked, forming a 1-cm-thick (0.4-in.-thick) puddle. This assumption provided a significant surface area from which to maximize the evaporation and formation of a toxic vapor cloud. Since the ALOHA model is limited by the maximum surface area of 31,400 $m^2$ (7.76 acres), for those chemical inventories that gave a 1-cm (0.4-in.) puddle greater than this limiting surface area, the calculated evaporation rate based on the limiting surface area of 31,400 $m^2$ (7.76 acres) was adjusted to reflect the actual inventory of the chemical and was modeled further as a direct source option.

For each of the identified chemicals in a gaseous state, the staff conservatively assumed that the entire contents were released over a ten-minute period as a continuous direct source. The results of these analyses were comparable or sometimes higher than were the applicant's results. The calculated concentrations of acetic acid and gasoline from water transport; gasoline and sodium hypochlorite from on-site storage, and 1-hexene, acetic acid, sodium chlorite, and ethylene exceeded IDLH concentrations at the outside of the control room. Because those concentrations pose a potential hazard to control room habitability, further analyses were required in Section 6.4, with the exception of 1-hexene from offsite storage at the OXEA Corporation.

The staff issued RAIs 02.02.03-6 and 02.02.03-7, pertaining to the analysis performed for 1-hexene. In its response dated November 20, 2008 (ML083290340), the applicant provided a reanalysis that considers a berm near the 1-hexene storage tank. This analysis demonstrates that the distance to IDLH (the temporary emergency exposure limit [TEEL]) concentration is 2,092 m (6,864 ft), which is well short of the 6,962 m (22,841 ft) to the control room. Based on the independent confirmatory calculations performed by the staff and documented in this section and in Section 6.4 of this SER, the staff found that none of the chemicals pose a threat to control room habitability. Therefore, RAIs 02.02.03-4 through 02.02.03-7, are resolved and closed.

*Fires*

The applicant considers accidents that could occur in the vicinity of the STP and could lead to high heat fluxes, smoke and nonflammable gas, or chemical-bearing clouds from the release of materials as a consequence of fires. The applicant considers and addresses fires in adjacent industrial plants, storage facilities, pipelines, brush and forest fires, and fires from transportation accidents. Based on review of the applicant's information, independent analyses performed by the staff regarding potential explosions and flammable vapor clouds, and a perception safety zone around STP, Units 3 and 4, the staff found that no hazardous effects are expected to affect the safe operation of STP, Units 3 and 4, from fires or heat fluxes associated with wild fires, fires in adjacent industrial plants, or from fires in on-site storage facilities.

*Collisions with Intake Structure*

The applicant addresses the effects of nearby navigable waterways with the intake structures. The staff reviewed the applicant's presented information. Based on a review of the information and consideration of a separate ultimate heat sink that provides water for safe shutdown and does not depend on this intake structure for makeup water, the staff found that potential damage to the Colorado River makeup water intake structure would not affect the safe shutdown of STP, Units 3 and 4.

*Liquid Spills*

The accidental release of oil or liquids that may be corrosive, cryogenic, or a coagulant may affect the safe shutdown of the plant if drawn into the plant's makeup water for the circulating water system.  However, a separate ultimate heat sink provides water for the safe shutdown and does not depend on the intake structure for makeup water for the safe shutdown of the plant.  Therefore, the staff found a spill will not have any effect on the safe shutdown of the plant.

*Radiological Hazards*

The control room habitability system for the ABWR provides the capability to detect and protect main control room personnel from airborne activity.  The ABWR control room is designed to withstand the effects of radiological events and consequential releases.

#### 2.2S.3.5 Post Combined License Activities

There are no post COL activities related to this section.

#### 2.2S.3.6 Conclusion

The staff reviewed the information in Section 2.2S.3, of the COL FSAR and found the applicant has identified potential accidents related to the presence of hazardous materials or activities in the site vicinity that could affect a nuclear power plant or plants of the specified type that might be constructed on the proposed site.  The applicant has also appropriately determined those that should be considered design-basis events and has demonstrated that the plant is adequately protected and can be operated with an acceptable degree of safety with regard to the DBAs.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations, the guidance in Section 2.2.3 of NUREG–0800, and other applicable RGs. The staff's review confirmed that the applicant has adequately addressed the COL License Information Items 2.7 and 2.8 in accordance with Section 2.2.3 of NUREG–0800, RG 1.91, the relevant requirements of CFR 52.79(a)(1)(iv), as they relate to the factors to be considered in the evaluation of sites, which require the location and description of industrial, military, or transportation facilities and routes, and the requirements of10 CFR 52.79(a)(1)(vi) as they relate to compliance with 10 CFR Part 100, and determined that no outstanding information is expected to be addressed in the COL FSAR related to this section.

The staff's review finds that the applicant has established that the construction and operation of a nuclear power plant or plants of the specified type on the proposed site location is acceptable.

## 2.3S Meteorology

To ensure that a nuclear power plant can be designed, constructed, and operated on an applicant's proposed site in compliance with the Commission's regulations, the staff evaluates regional and local climatological information, including climate extremes and severe weather occurrences that may affect the design and siting of a nuclear plant.  The staff reviews information on the atmospheric dispersion characteristics of a nuclear power plant site to determine whether the radioactive effluents from postulated accidental releases, as well as routine operational releases, are within the Commission's guidelines.

## 2.3S.1 Regional Climatology

### *2.3S.1.1 Introduction*

This section of the FSAR addresses the averages and extremes of climatic conditions and regional meteorological phenomena that could affect the safe design and siting of the plant. The information describes the general climate, severe weather phenomena, meteorological data for evaluating the ultimate heat sink (UHS), design-basis dry- and wet-bulb temperatures, restrictive dispersion conditions, and climate change.

### *2.3S.1.2 Summary of Application*

This site-specific supplement in the FSAR describes the following:

- Data sources used to characterize the regional climatological conditions pertinent to the proposed site.
- The general climate of the region with respect to types of air masses, synoptic features (high- and low-pressure systems), general airflow patterns (wind direction and speed), temperature and humidity, and precipitation (rain, snow, freezing rain, and sleet).
- Frequencies and descriptions of severe weather phenomena that have affected the proposed site including extreme winds, tornadoes, tropical cyclones, precipitation extremes, hail, freezing rain, sleet, winter precipitation (snow), thunderstorms, and lightning.
- Meteorological conditions for evaluating the UHS.
- Design-basis dry- and wet-bulb temperatures for the proposed site.
- The potential for restrictive air dispersion conditions and high air pollution levels at the proposed site.

In addition, in FSAR Section 2.3S.1.5, the applicant provides the following:

*Tier 1 Departure*

- STP DEP T1 5.0-1 Site Parameter

The applicant identifies one-percent maximum coincident and noncoincident wet-bulb temperatures and the zero-percent maximum noncoincident wet-bulb temperature as departures from ABWR DCD Tier 1, Table 5.0, “ABWR Site Parameters”; and Tier 2, Table 2.0-1, “Envelope of ABWR Standard Plant Site Design Parameters.”

*COL License Information Item*

- COL License Information Item 2.1 Non-Seismic Design Parameters

This site-specific supplement addresses COL License Information Item 2.1, from the certified DCD, which states that “compliance with the envelope of standard plant site non-seismic design

parameters of DCD Tier 2, Table 2.0-1 shall be demonstrated for design-bases events."
DCD Tier 2, Section 2.2.1, further states that for design-basis events, the site is acceptable if all of the site characteristics fall within the envelope of ABWR standard plant site design parameters given in DCD Tier 2, Table 2.0-1. For cases where a characteristic exceeds its envelope, it will be necessary for the COL applicant to submit analyses to demonstrate that the overall set of site characteristics does not exceed the capability of the design. The DCD Tier 2, Table 2.0-1 envelope of ABWR standard plant site design parameters includes extreme wind, tornado, precipitation (for roof design), maximum snow load, and ambient design temperature site parameters.

### *2.3S.1.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the regional climatology, and the associated acceptance criteria, are in Section 2.3.1, "Regional Climatology," of NUREG–0800. In particular, the regulatory requirements are 10 CFR 52.79(a)(1)(iii), 10 CFR 100.20(c)(2) and 100.21(d).

The staff considered the following regulatory requirements in reviewing the applicant's discussion of regional climatology:

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

- 10 CFR 100.20(c) (2) and 100.21(d), with respect to consideration of the regional meteorological characteristics of the site.

NUREG–0800, Section 2.3.1 specifies that an application meets the above requirements if the application satisfies the following criteria:

- The description of the general climate of the region should be based on standard climatic summaries compiled by the National Oceanic and Atmospheric Administration (NOAA). Consideration of the relationships between regional synoptic-scale atmospheric processes and local (site) meteorological conditions should be based on appropriate meteorological data.

- Data on severe weather phenomena should be based on standard meteorological records from nearby representative National Weather Service (NWS), military, or other stations recognized as standard installations that have long periods of data on record. The applicability of these data to represent site conditions during the expected period of reactor operation should be substantiated.

- The tornado parameters should be based on RG 1.76, Revision 1, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants." Alternatively, an applicant may specify any tornado parameters that are appropriately justified, provided that a technical evaluation of site-specific data is conducted.

- The straight-line wind speed site characteristics should be based on appropriate standards, with suitable corrections for local conditions.

- UHS meteorological data, as stated in RG 1.27, Revision 2, "Ultimate Heat Sink for Nuclear Power Plants (For Comment)," should be based on long-period regional records that represent site conditions.

- The 100-year, ground-level snowpack or snowfall, whichever is greater, should be based on data recorded at nearby representative climatic stations or obtained from appropriate standards with suitable corrections for local conditions. The 48-hour probable maximum winter precipitation (PMWP) should be determined in accordance with reports published by NOAA's Hydrometeorological Design Studies Center.

- Ambient temperature and humidity statistics should be derived from data recorded at nearby representative climatic stations or obtained from appropriate standards with suitable corrections for local conditions.

- Information depicting the potential for high air pollution levels should be based on U.S. Environmental Protection Agency (EPA) studies.

- All other meteorological and air quality conditions identified by the applicant as design and operating bases should be documented and substantiated.

Generally, the information should be presented and substantiated in accordance with acceptable practices and data promulgated by NOAA, industry standards, and RGs.

Subsequent to the publication of SRP Section 2.3.1, the staff issued proposed interim staff guidance (ISG) document DC/COL-ISG-7, "Interim Staff Guidance on Assessment of Normal and Extreme Winter Precipitation Loads on the Roofs of Seismic Category I Structures," for public comment on August 22, 2008 (73 *FR* 49712) (ML081980084). The purpose of the document is to clarify the staff's position on identifying winter precipitation events as site characteristics and site parameters for determining normal and extreme winter precipitation loads on the roofs of seismic Category I structures. The final version of DC/COL-ISG-7 was issued on July 1, 2009 (74 *FR* 31470) (ML091490565).

To the extent that the data are applicable to the acceptance criteria outlined above, the applicant applies the following NRC-endorsed meteorological information selection methodologies and techniques:

- RG 1.23, Revision 1, "Meteorological Monitoring Programs for Nuclear Power Plants," provides criteria for an acceptable onsite meteorological measurements program that can be used to monitor regional meteorology site characteristics.

- RG 1.27, provides criteria for selecting the UHS meteorological data that would result in maximum evaporation and drift loss of water and minimum water cooling.

- RG 1.76, provides criteria for selecting the design-basis tornado parameters.

- RG 1.206, describes the type of regional meteorological data that should be in FSAR Tier 2, Section 2.3S.1.

- RG 1.221, “Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants,” provides criteria for selecting the design-basis hurricane wind speed.

When independently assessing the veracity of the information presented by the applicant in FSAR Tier 2, Section 2.3S.1, the staff applied the same methodologies and techniques cited above.

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 related to this SER section. Tier 1 departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4.

### *2.3S.1.4 Technical Evaluation*

The staff reviewed the application and the applicant’s responses to the RAIs to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding regional climatology. The staff followed the procedures described in Section 2.3.1 of NUREG-0800, as part of the review.

The staff reviewed the following information in the COL FSAR:

*Tier 1 Departure*

In general, the Tier 1 Departure identified by the applicant in this section requires prior NRC approval in the form of an exemption and the full scope of their technical impact may be evaluated in the other sections (and chapters) of this SER. For more information, refer to COL application Part 7, Section 5.0 for a listing of all FSAR sections affected by this Tier 1 departure.

- STP DEP T1 5.0-1 Site Parameter

This departure is evaluated in Section 2.3S.1.4.5 of this SER.

*COL License Information Item*

- COL License Information Item 2.1 Non-Seismic Design Parameters

The staff’s review of the climatological “Non-Seismic Design Parameters” (i.e., extreme wind, tornado, precipitation [for roof design], maximum snow load, and ambient design temperature site parameters) is summarized below.

2.3S.1.4.1 Data Sources

The applicant characterizes the regional climatology of the proposed STP, Units 3 and 4, site using data from the National Climatic Data Center (NCDC); including the first order NWS station in Victoria, Texas, and 14 other nearby cooperative observation stations. All of these observation stations are located in the Texas Upper Coast climatic division (TX-8) except for the Aransas Wildlife Refuge observation station, which is in the Texas South Central climatic division (TX-7). The regional climatic observation stations used by the applicant are listed in FSAR Tier 2, Table 2.3S.1-1.

The applicant stated that the selection criteria used for the observation stations include the following:

- Proximity to the STP site (i.e., within an approximate 50-km [31.25 mi] radius).
- Coverage in all directions surrounding the site (to the extent possible).
- Selection of a station if it contributed one or more extreme conditions (e.g., rainfall, snowfall, maximum and/or minimum temperatures) for that given direction relative to the site where more than one station exists for a given direction.

The applicant also states that if an overall extreme precipitation or temperature condition was identified for a station located within a reasonable distance beyond the 50-km (31.1-mi) radius, and that extreme condition was considered to be reasonably representative of the site area, that station was also included.

The applicant also obtained information on mean and extreme regional climatological phenomena from a variety of sources, such as publications by the NOAA, NCDC, American Society of Civil Engineers (ASCE), Structural Engineering Institute (SEI), American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE), NOAA Air Resources Laboratory, NOAA Coastal Services Center (CSC), NOAA National Severe Storms Laboratory (NSSL), U.S. Department of Agriculture Pacific Wildland Fire Sciences Lab, and the U.S. Department of Agriculture Rural Utilities Service.

The staff found the applicant's sources for regional climatological data to be appropriate because the sources include NOAA and industry standards, as specified in SRP Section 2.3.1.

2.3S.1.4.2 General Climate

The applicant describes the general climate of the proposed STP site as maritime subtropical (or humid subtropical), which is characterized by mild, short winters; long periods of mild sunny weather in autumn; windy but mild weather in spring; and long hot summers. Maritime tropical air mass characteristics prevail much of the year, especially during the summer with the establishment of the Bermuda High and the Gulf of Mexico High. This circulation pattern is occasionally disrupted by the passage of synoptic- and meso-scale weather systems during the transitional seasons (spring and autumn) and winter months. During winter, cold air masses originating in the continental interior around Colorado or Canada may briefly intrude into the region. These systems may result in a variety of precipitation events that include rain, sleet, and/or freezing rain. Larger persistent outbreaks of very cold, dry air associated with massive high-pressure systems that move southward out of Canada also occasionally affect the site region. However, these weather conditions tend to be modified significantly as land modification warms the cold air that reaches the proposed STP site.

The applicant stated that monthly precipitation exhibits a cyclical pattern, with the predominate maximum occurring in May and a secondary maximum occurring in September. Strong winds associated with tropical cyclones can have a significant effect on the site area due to its proximity to the Gulf of Mexico.

The staff agreed with the applicant's description of the general climate of the region. The staff relied on the NCDC narrative, "Local Climatological Data, Annual Summary with Comparative

Data for Victoria, Texas," to reach this conclusion. The staff issued RAI 02.03.01-1, requesting the applicant to discuss the influence of the Gulf of Mexico and the resulting land and sea breezes on regional climatology. In its response to RAI 02.03.01-1, dated May 29, 2008 (ML081560317), the applicant stated that the land/sea temperature contrast during summer days creates a circulation forming a sea breeze where cooler, more saturated air pushes inland as the warm air rises inland. The opposite occurs at night, where inland plains cool rapidly while the sea stays relatively warmer, thus causing a breeze to push off-shore into the Gulf of Mexico. The applicant has incorporated this information into Revision 3 of the FSAR. Therefore, RAI 02.03.01-1 is resolved and closed.

### 2.3S.1.4.3 Severe Weather

#### 2.3S.1.4.3.1 Extreme Winds

ABWR DCD Tier 2, Section 3.3.1, states that the design wind pressures and forces for buildings, components and cladding, and other structures at various heights above the ground were obtained in accordance with ASCE 7–88, "Minimum Design Loads for Buildings and Other Structures." Figure 1 of ASCE 7–88, provides a plot of "basic wind speeds" for the contiguous states, which it defines as the fastest-mile wind speed at 10 m (33 ft) above the ground for terrain Exposure Category C [1] and associated with an annual probability of occurrence of 0.02 (i.e., 50-year mean recurrence interval). To account for the degree of hazard to human life and damage to property, ASCE 7-88 suggests scaling these fastest-mile basic wind speeds by an importance factor of 1.11 for essential facilities located at hurricane coastlines, which accounts for an increase in the recurrence interval from 50 to 100 years.

As described in ABWR DCD Tier 2, Section 3.3.1, the basic wind speeds used for the ABWR wind loading design are 177 km per hour (km/h) (110 miles per hour [mph]) with a recurrence interval of 50 years, and 197 km/h (122 mph) with a recurrence interval of 100 years. ABWR DCD Tier 1, Table 5.0 and Tier 2, Table 2.0-1 identify these wind speed values as extreme wind "basic wind speed" site design parameters, with further clarifications that the 177 km/h (110 mph) value is used for the design of non-safety-related structures and the 197 km/h (122 mph) value is used for the design of safety-related structures. The COL license information item in ABWR DCD Tier 2, Section 3.3.3.1, states that the site-specific, design-basis wind shall not exceed these design-basis wind parameters.

A more recent 2005 version of ASCE 7–88, ASCE/SEI 7–05, incorporated substantial changes in defining wind loads, including: (1) redefining the basic wind speed as the three-second gust speed (instead of the fastest-mile speed) at 10 m (33 ft) above the ground in exposure Category C, and (2) revising the map of basic wind speeds to reflect a newer analysis of hurricane wind speeds. The applicant defines the STP extreme wind basic wind speed site characteristics as three-second gusts using a linear interpolation from the map of basic wind speeds in ASCE/SEI 7–05 for the portion of the United States that includes the proposed STP site. The ASCE/SEI 7–05 plot of three-second gust basic wind speeds is associated with a mean recurrence interval of 50 years. Using this plot, the applicant defined the 50-year return period three-second gust basic wind speed for the proposed STP site as 201 km/h (125 mph). Using a conversion factor of 1.07, which is listed in Table C6-3 of ASCE/SEI 7–05 as the ratio of the peak gust wind speed 100-year to 50-year mean recurrence interval values, the applicant

[1] ASCE 7-88 defines Exposure C as open terrain with scattered obstructions having heights generally less than 30 feet, including open country and grasslands.

derived a 100-year return period 3-second gust basic wind speed site characteristic value of 215 km/h (134 mph).

The staff notes that according to Table C6-2 of ASCE/SEI 7–05, the applicant's 100-year return period of a three-second gust basic wind speed site characteristic value of 215 km/h (134 mph) is equivalent to a Saffir-Simpson Category 3 hurricane. A discussion on the occurrence of tropical cyclones in the STP site region is in FSAR Tier 2, Subsection 2.3S.1.3.3.

In order to compare the ABWR fastest-mile basic wind speed site design parameters to the STP's three-second gust basic wind speed site characteristics, the applicant converted the ABWR fastest-mile basic wind speed site design parameter values to the equivalent of three-second gust wind speed values. The applicant stated that the ABWR fastest-mile extreme wind basic wind speed site design parameters of 177 km/h (110 mph) and 197 km/h (123 mph) convert to three-second gust values of 203 km/h (126 mph) and 224 km/h (139 mph), respectively. The staff performed a similar conversion using the relationship among wind speed averaging times shown in Figure C6-4 of ASCE/SEI 7–05 and obtained similar results (e.g., within 1.6 km/h [1 mi/h]). This conversion demonstrates that the ABWR extreme wind basic wind speed standard plant site design parameters bound the corresponding extreme wind site characteristics chosen by the applicant, thus satisfying COL License Information Item 2.1, with respect to extreme winds.

In FSAR Tier 2, Revision 0, Section 2.3S.1.3.3, states that one Category 5, four Category 4, and nine Category 3 hurricanes were reported by NOAA-CSC to have tracked within a 185-km (100-nautical mile [nmi]) radius of the STP, Units 3 and 4, sites during the period from 1851 to 2006. Using this same NOAA-CSC database for this same period of record, the staff identified 11 hurricanes that were classified as major (i.e., Saffir-Simpson Category 3 or higher) at the time they may landfall within 185 km (100 nm) of the STP site. For each of these 11 major hurricanes, the staff used the sustained wind speeds reported in the NOAA-CSC database at landfall along with information presented in Table C6-2 of ASCE/SEI 7–05, to estimate the corresponding three-second gust wind speed over land at landfall. Because hurricane wind speeds typically decrease as storms move inland, and the STP site is located approximately 24 km (15 mi) inland from the Gulf of Mexico, the staff reduced the gust wind speed at landfall by 8 km/h (5 mi/h) based on the 8 km/h (5 mph) reduction in basic wind speed from the coastline to the inland location of the STP site, as shown in Figure 6-1A of ASCE/SEI 7–05. The staff found that 8 out of the 11 major landfall hurricanes had projected gust wind speed values that exceeded the applicant's selected extreme wind basic wind speed site characteristic value of 215 km/h (134 mph) for safety-related structures. The highest gust wind speed of 297 km/h (184 mph) was associated with an unnamed storm in August 20, 1886. The staff subsequently issued RAI 02.03.01-4, requesting the applicant to justify why the extreme wind basic wind speed site characteristic value for safety-related structures is not based on the most severe hurricanes historically reported for the site and the surrounding area.

In its response to RAI 02.03.01-4, dated May 29, 2008 (ML081560702), the applicant stated that it provided the 100-year return period 3-second gust wind speed as the extreme wind basic wind speed site characteristic value for consideration in evaluating the design and operation of the proposed facility, in accordance with RG 1.206, Regulatory Position C.I.2.3.1.2. Furthermore, the applicant stated that the 100-year return period 3-second gust wind speed site characteristic value was determined in accordance with the acceptance criteria specified in SRP Section 2.3.1.

In a follow-up to the applicant's response to RAI 02.03.01-4, the staff issued RAI 02.03.01-21, requesting the applicant to revise the FSAR to identify the extreme wind basic wind speed site characteristic value for the STP site and surrounding area based on the most severe hurricanes historically reported for that area. 10 CFR 52.79(a) (iii) states (in part) that the COL FSAR shall include the meteorological characteristics of the proposed site with an appropriate consideration of the most severe of the natural phenomena historically reported for the site and surrounding area and with a sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated. In order to be compliant with 10 CFR 52.79(a)(iii), the staff believed the extreme wind basic wind speed site characteristic value for the STP site and surrounding area should consider the most severe hurricanes historically reported for the STP site and surrounding area.

In its response to RAI 02.03.01-21, dated May 26, 2009 (ML091490166), the applicant proposed a revision to extreme winds in FSAR Tier 2, Subsection 2.3S.1.3.1. The proposed revision repeats the previous statements that the design extreme wind loading is based on a basic wind speed, which is the 3-second gust at 10 m (33 ft) above the ground in Exposure Category C, as defined in ASCE/SEI 7–05. The proposed revision also states that the applicant has reviewed the NOAA CSC historical record of tropical cyclone tracks and intensities near the STP, Units 3 and 4, sites from 1851 to the present. This review identifies eleven tropical cyclones with wind speeds that exceed a design-basis extreme wind loading for the STP, Units 3 and 4, sites calculated in accordance with ASCE/SEI 7–05. The applicant further stated that the wind speeds identified during this review were bounded by the 322 km/h (200 mph) maximum tornado wind speed site characteristic value: therefore, those speeds do not represent a threat to the integrity of any STP, Units 3 and 4, safety-related structures, systems, and components (SSCs).[1]

Initially, the U.S. Atomic Energy Commission (predecessor to the NRC) considered tornadoes to be the bounding extreme wind events and issued the original version of RG 1.76, dated April 1974. The selection of the design-basis tornado wind speeds was premised on the probability that a tornado exceeding those speeds would be on the order of $10^{-7}$ per year per nuclear power plant. In March 2007, the NRC issued Revision 1 of RG 1.76, which relied on the Enhanced Fujita Scale that was implemented by the NWS, dated February 2007. The Enhanced Fujita Scale is a revised assessment relating tornado damage to wind speed, which resulted in a decrease in design-basis tornado wind speed criteria in Revision 1 of RG 1.76. Because the design-basis tornado wind speeds decreased as a result of the analysis that was performed to update RG 1.76, it was no longer clear that the revised tornado design-basis wind speeds would bound design-basis hurricane wind speeds in all areas of the United States. This uncertainty prompted an investigation into extreme wind gusts during hurricanes and their relation to design-basis hurricane wind speeds, which resulted in issuing RG 1.221, dated October 2011. RG 1.221 defines the design-basis hurricane as having the same $10^{-7}$ per year exceedance frequency as the design-basis tornado. The staff subsequently issued RAI 02.03.01-24, requesting, in part, that the applicant identify a design-basis hurricane wind speed for the STP site, given that RG 1.221 describes a method that the staff considers acceptable for selecting site-specific design-basis hurricane wind speeds.

[1] The "extreme wind basic wind speed" and the "maximum tornado wind speed" site parameters are used with different load factors and load combinations in the ABWR DCD to evaluate the capacity of SSCs to withstand wind pressures.

In its response to RAI 02.03.01-24, dated January 12, 2012 (ML12018A387), the applicant identifies a STP site-specific design-basis hurricane wind speed of 338 km/h (210 mph) for a three-second gust wind speed, based on the guidance in RG 1.221. To ensure that the STP, Units 3 and 4, design reflects the guidance in RG 1.221, the applicant revised FSAR Tier 2 Revision 8, Table 2.0-2, “Comparison of ABWR Standard Plant Site Design Parameters and STP 3 & 4 Site Characteristics,” to include 338 km/h (210 mph) as a site-characteristic hurricane wind speed for STP, Units 3 and 4.

The staff confirmed that the applicant’s 338 km/h (210 mph) site-specific design-basis hurricane wind speed derived from RG 1.221 is correct. Because the highest historic hurricane gust wind speed projected by the staff (297 km/h [184 mph]) as discussed above is bounded by the 338 km/h (210 mph) hurricane wind speed site characteristic value that the applicant identified, the staff found the applicant’s response to RAI 02.03.01-24 acceptable. In addition, the applicant has committed in its response to RAI 02.03.01-24 to ensuring that the STP, Units 3 and 4, design reflects this 338 km/h (210 mph) hurricane wind speed site characteristic value. Therefore, RAIs 02.03.01-4, 02.03.01-21, and 02.03.01-24 are resolved and closed.

#### 2.3S.1.4.3.2 Tornadoes

The staff issued RAI 02.03.01-2, requesting the applicant to provide statistics on the frequency of tornadoes in the STP site region. In its response to RAI 02.03.01-2, dated May 29, 2008 (ML091490166), the applicant identifies 902 tornado occurrences in the counties that are either totally or partially within a 125.5-km (78-mi) radius of the STP site.[1] The applicant uses the NCDC Storm Events database for the period January 1950 through August 2006. The applicant has incorporated this information into Revision 3 of the FSAR. The staff reviewed the same NCDC database for the period January 1950, through April 2008, and identified a slightly lower number (823) of tornado occurrences for this same region. Because the applicant’s estimated tornado frequency bounds the staff’s estimated tornado frequency, RAI 02.03.01-2 is resolved and closed.

NUREG/CR–4461, Revision 2, “Tornado Climatology of the Contiguous United States,” provides the basis for the design-basis tornado wind speed in Revision 1 to RG 1.76. Appendix C to NUREG/CR–4461, contains estimates of strike probabilities by one-degree latitude and longitude boxes. The STP is located about N 28.8 degree latitude and W 96.1 degree longitude, near the corners of four of these one-degree boxes. The average expected strike probability per year among these four one-degree boxes (weighted by the faction each one-degree box area is assumed to be covered by land) for a structure with a characteristic dimension of 61 m (200 ft) is 1.75E-04, which corresponds to a mean recurrence interval of approximately 5,710 years.

ABWR DCD Tier 2, Section 3.3.1, “Design Wind Velocity,” states that the design-basis tornado is described (in part) by the following parameters:

- A maximum tornado wind speed of 483 km/h (300 mph) at a radius of 45.7 m (150 ft) from the center of the tornado.
- A maximum translational velocity of 97 km/h (60 mph).

[1] According to the applicant, the 125.5-km (78-mi) radius covers the same area as a 2-degree longitude-by-latitude box surrounding the STP site.

- A maximum tangential velocity of 386 km/h (240 mph) based on the translational velocity of 97 km/h (60 mph).

- A maximum atmospheric pressure drop of 13.8 kPa (2 psi) with a rate of the pressure change of 8.3 kPa per second (kPa/s) (1.2 psi per second [psi/s]).

These design-basis tornado parameters are listed in ABWR DCD Tier 2, Table 2.0-1 as standard plant site design parameters; the maximum tornado wind speed and maximum pressure drop parameters are listed in ABWR DCD Tier 1, Table 5.0 as site parameters.

The applicant chose the tornado site characteristics based on Revision 1 to RG 1.76. RG 1.76 provides design-basis tornado characteristics for three tornado intensity regions throughout the United States, each with a $10^{-7}$ per year probability of occurrence. The proposed STP site is located in tornado-intensity Region II. The applicant has chosen to use the design-basis tornado characteristics from Region II and, correspondingly, proposes the following tornado site characteristics:

- A maximum wind speed of 322 km/h (200 mph).

- A translational speed of 64 km/h (40 mph).

- A maximum rotational speed of 257 km/h (160 mph).

- The radius of a maximum rotational speed of 45.7 m (150 ft).

- A pressure drop of 6.2 kPa (0.9 psi).

- A rate of pressure drop of 2.8 kPa/s (0.4 psi/s).

Because the applicant's design-basis tornado site characteristics are based on RG 1.76, the staff concluded that the applicant has chosen acceptable tornado site characteristics.

FSAR Tier 2, Table 2.0-2, compares the ABWR tornado site parameters to the STP, Units 3 and 4, tornado site characteristics. Because the ABWR tornado standard plant site design parameters bound the corresponding STP tornado site characteristics, COL License Information Item 2.1, with respect to tornadoes, is resolved.

2.3S.1.4.3.3 Tropical Cyclones

In FSAR Tier 2, Revision 0, Section 2.3S.1.3.3, states that during the period between the years 1851, and 2006, 142 tropical cyclones centers or storm tracks passed within a 185-km (100-nmi) radius of the proposed STP, Units 3 and 4, site. The applicant used the NOAA-CSC historical tropical database to derive these results. Using the same database, the staff was able to verify 75 tropical cyclones centers or storm tracks passed within a 185-km (100-nmi) radius of the proposed STP site.

The staff also reviewed the tropical cyclone reports published by the NWS National Hurricane Center (NHC) between the years 2007 and 2008 to determine whether any additional tropical cyclones tracked within a 185-km (100-nmi) radius of the proposed STP site with hurricane force winds during this time period. The staff found that Hurricane Ike made landfall along the upper Texas coast at the upper end of Category 2 intensity in September 2008.

“Major hurricane” is a term utilized by NHC for hurricanes that reach maximum sustained 1-minute surface winds of at least 179 km/h (111 mph). This speed is equivalent to at least a Category 3 hurricane on the Saffir-Simpson scale. The NOAA-CSC database shows that a total of 11 major hurricanes have impacted the 185-km (100-nmi) area surrounding the proposed STP site between the years 1851 and 2006. These data translate to a reoccurrence rate of 0.07 per year, or one major hurricane every 14.2 years.

Tropical systems can also cause significant amounts of rainfall. The applicant reports that one-third of the individual 24-hour rainfall records were associated with tropical cyclones that passed within a 185-km (100-nmi) radius of the STP site. The staff independently confirmed these statistics.

The staff issued RAI 02.03.01-16, requesting the applicant to confirm the number of tropical cyclone storm tracks that have passed near the STP site and to revise, as necessary, FSAR Tier 2, Section 2.3S.1.3.3.

In its response to RAI 02.03.01-16, dated December 18, 2008 (ML083570395), the applicant stated that a recount of the tropical cyclone inventory taken from the NOAA-CSC database produces statistics similar to those compiled by the staff. The applicant reports that 1, Category ; 6, Category 4; 4 Category 3; 5 Category 2; and 22 Category 1 hurricane tracks (on the Saffir-Simpson Hurricane scale) have passed within a 185-km (100-nmi) radius of the STP site between the years 1851 and 2006. The applicant includes these revised tropical cyclone statistics in Revision 3 to the FSAR. Therefore, RAI 02.03.01-16 is resolved and closed.

2.3S.1.4.3.4 Precipitation Extremes

This discussion is intended to provide a general climatic understanding of the severe weather phenomena in the site region. However, the discussion does not generate site characteristics for use as design or operating bases.

The applicant uses historical climate data from 15 nearby observation stations, which are listed in FSAR Tier 2, Table 2.3S-1, “NWS and Cooperative Observing Stations Near the STP 3 & 4 Site,” to identify precipitation extremes (rainfall and snowfall) observed near the proposed STP site. Based on the distribution of the observation stations around the site, these data can be used to adequately represent precipitation extremes that might be expected to occur at the site.

Although some of the recorded precipitation extremes are associated with the occurrence of tropical cyclones, the overall highest 24-hour rainfall total is not. On October 19, 1983, the 24-hour rainfall record in the area surrounding the proposed STP site was set at the Bay City Waterworks, when 53 cm (20.85 in.) fell. The overall highest monthly total, 80.3 cm (31.61 in.) during September 1979, at Freeport 2NW observation station, was partially attributed to Tropical Storm Elena.

The applicant stated that snow accumulation is a rare occurrence in the vicinity of the STP site. According to the applicant, most winters bring no accumulation of snowfall and storms that produce large measurable amounts of snow are rare. The staff’s review of the NCDC Daily Surface Snowfall data for the 15 climatic stations listed in FSAR Tier 2, Table 2.3S-3, “Climatological Extremes at Selected NWS and Cooperative Observing Stations in the STP 3 & 4 Site Area,” indicates that average daily snowfall totals equal to or greater than 2.54 cm (1 in.) are recorded once every 14 years. A Christmas storm in 2004, was responsible for the highest overall 24-hour and monthly snowfall totals recorded in the site region—26.7 cm

(10.5 in.) at the Davevang 1W observation station—located approximately 32 km (19.9 mi) north-northwest of the STP site. The applicant stated that it is reasonable to assume that this snowfall did not remain for more than a few days, because the high temperatures for the following few days exceeded the freezing mark.

The staff concluded that the applicant has adequately identified precipitation extremes that might be expected to occur in the vicinity of the site. FSAR Tier 2, Table 2.3S-3, lists the highest precipitation extremes in the vicinity of the site.

In FSAR Tier 2, Table 2.3S-3, the applicant provides climatic extremes for each of the utilized observation stations (when available), including maximum 24-hour and monthly rainfall and snowfall. The staff independently verified these rainfall records using the NCDC Climate Data Online Daily (TD3200/3210) and Monthly Surface Data (DS-3220). The staff found some discrepancies and issued RAI 02.03.01-18, requesting the applicant to confirm several of the rainfall statistics in FSAR Tier 2, Table 2.3S-3. In its response to RAI 02.03.01-18, dated December 18, 2008, the applicant revised several of the rainfall statistics in Revision 3 of the FSAR. Therefore, RAI 02.03.01-18 is resolved and closed.

2.3S.1.4.3.5 Hail, Freezing Rain, and Sleet

The following discussion on hail, freezing rain, and sleet is intended to provide a general climatic understanding of the severe weather phenomena in the site region. However, the discussion does not generate site characteristics for use as design or operating bases.

The online NWS Glossary defines hail as showery precipitation in the form of irregular pellets or balls of ice, more than 5 millimeters (mm) (0.2 in.) in diameter, falling from a cumulonimbus or thunderstorm cloud. Hail generally occurs during the spring and can be a major weather hazard that causes significant damage to crops and property.

The applicant used the NOAA “Climate Atlas of the United States” CD-ROM to estimate that around the proposed STP site area, the annual mean number of days with hail of 19 mm (0.75 in.) or greater in diameter is approximately one day per year. The applicant also used the online NCDC Storm Event Database for Texas to identify the maximum hail events observed in Matagorda County and surrounding counties. The applicant stated that the maximum diameter of hail observed in Matagorda County is approximately 50.8 mm (2 in.) Hailstorm events for surrounding counties have reported maximum hail stone diameters ranging between 50.8 to 114.3 mm (2.0 and 4.5 in.) The applicant stated that hail the size of grapefruit (approximately 114.3 mm [4.5 in.] in diameter) was observed on two occasions at two different locations in the general STP site area: (1) on April 11, 1995, in Calhoun, Texas (in Calhoun County), approximately 108 km (67 mi) north-northwest of the STP site; and (2) on June 20, 1996, in Egypt, Texas (in Wharton County), approximately 69 km (43 mi) north-northwest of the STP site. The staff noted that NOAA’s National Severe Storms Laboratory’s Severe Thunderstorm Climatology Web site reports that, on average, there are three to four days per year with hail at least 19 mm (0.75 in.) in diameter and one-fourth to one-half days per year with hail at least 50.8 mm (2 in.) in diameter occurring within 40 km (25 mi) of the STP site.

Sleet is defined as pellets of ice composed of frozen or mostly frozen raindrops or refrozen, partially melted snowflakes that usually bounce after hitting the ground or other hard surfaces. Freezing rain is defined as rain that falls as a liquid but freezes into a glaze upon contact with the ground. Depending on the temperature characteristics of the air mass, snow events are often accompanied by or alternate between sleet and freezing rain. The applicant stated that according to the NOAA "Climate Atlas of the United States" CD-ROM, freezing precipitation occurs approximately 2.5 to 5.4 days per year at the STP site.

The applicant also states that there have been no reported records of probable annual frequency of dust storms at the STP site area. The staff expects that dust and sand storms would be a rare occurrence due to the abundance of ground vegetation in the STP site region.

The staff verified the hail and freezing precipitation statistics presented by the applicant by reviewing the NCDC online "Climatic Atlas of the United States" and "Storm Event Database for Texas." In Technical Report 2002-01, "The Development of a U.S. Climatology of Extreme Ice Loads," the NCDC also reports a 50-year return period uniform radial ice thickness of 12.7 mm (0.5 in.) because of freezing rain, with a concurrent 3-second gust wind speed of 48 km/h (30 mph) for the proposed STP site area.

2.3S.1.4.3.6 Winter Precipitation Loads

Section 2.3.1 of NUREG–0800, states that the winter precipitation loads included in the combination of normal live loads considered in the design of a nuclear power plant that might be constructed on a proposed COL site should be based on the weight of the 100-year snowpack or snowfall, whichever is greater, recorded at ground level. Likewise, the winter precipitation loads included in the combination of extreme live loads considered in the design of a nuclear power plant that might be constructed on a proposed COL site, should be based on the weight of the 100-year snowpack at ground level plus the weight of the 48-hour PMWP at ground level, for the month corresponding to the selected snowpack. A COL applicant may choose to justify an alternative method for defining the extreme winter precipitation load by demonstrating that the 48-hour PMWP could neither fall on nor remain on top of the snowpack and/or building roofs.

In FSAR Tier 2, Section 2.3S.1.3.4, the applicant stated that the evaluation of normal and extreme live snow loads on the roofs of safety-related structures does not appear to be warranted for STP, Units 3 and 4, because of the infrequent occurrence of snowfall events, and the fact that snowfall events do not appear to persist for any appreciable period of time as ground level snowpack. Consequently, the applicant identifies a 100–year return period value for ground level snowpack at zero-pound force per square foot ($lbf/ft^2$) for the proposed STP site, which is in accordance with ASCE/SEI 7–02.

The staff issued RAI 02.03.01-5, requesting the applicant to explain why the maximum snow load site characteristic value is not based on the highest snowfall value historically reported for the site and the surrounding area. In its response to RAI 02.03.01-5, dated May 29, 2008 (ML081560702), the applicant stated that the highest snowfall value historically reported for the site vicinity was 26.7 cm (10.5 in.) of snow recorded at Danevang 1W on December 25, 2004. Using a water-equivalent ratio of 10 percent, the applicant estimated that this 26.7-cm (10.5-in.) snowfall had a liquid water equivalent of 2.7 cm (1.05 in.), which is equal to a weight of 0.263 kPa (5.5 pounds force per square foot [$lbf/ft^2$]). The applicant lists 0.263 kPa (5.5 $lbf/ft^2$) as the maximum ground level snow load in Revision 3 to FSAR Tier 2, Table 2.0-2. The staff found this response acceptable and RAI 02.03.01-5 is resolved and closed.

Also, the applicant did not identify a 48-hour PMWP value for the STP site in Revision 0 to the FSAR. Consequently, the staff issued RAI 02.03.01-6, requesting the applicant to identify a

48-hour PMWP site characteristic value for the STP site and to describe the additional resulting weight on the roof if all the roof drains are clogged by snow and/or ice. In its response to RAI 02.03.01-6, dated May 29, 2008 (ML081560702), the applicant identified a 48-hour PMWP of 86.4 cm (34 in.) of liquid precipitation based on an interpolation of data in NUREG/CR–1486, "Hydrometeorological Report No. 53, Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105$^{th}$ Meridian." The staff performed an independent 48-hour PMWP evaluation using the NUREG/CR–1486 data. The staff obtained similar results (i.e., within three percent). Because the applicant had determined this value in accordance with NUREG/CR–1486, the staff concluded that a 48-hour PMWP site characteristic value of 86.4 cm (34 in.) of water is acceptable.

In its response to RAI 02.03.01-6 (ML081560702), dated May 29, 2008, the applicant stated that the standard ABWR seismic Category I structures have roofs without parapets or parapets with scuppers to supplement roof drains, so that large inventories of water cannot accumulate. The applicant also notes that the roof structure of the site-specific seismic Category I structures (i.e., reactor service water pump houses) are designed without parapets so that excessive ponding of water cannot occur. Therefore, RAI 02.03.01 is resolved and closed.

The staff issued RAI 02.03.01-14, requesting the applicant to revise FSAR Tier 2, Section 2.3S.1, to identify the normal winter precipitation event, the extreme frozen winter precipitation event, and the extreme liquid precipitation event as site characteristics in accordance with DC/COL-ISG-7.

The staff issued the proposed DC/COL-ISG-7 for public comment on August 22, 2008 (73 *FR* 49712). The intent was to clarify the staff's position on identifying winter precipitation events as site characteristics and site parameters for determining normal and extreme winter precipitation loads on the roofs of seismic Category I structures. DC/COL-ISG-7 revises the previously issued staff guidance discussed in SRP Section 2.3.1.

DC/COL-ISG-7, states that normal and extreme winter precipitation events should be identified in SRP Section 2.3.1 as COL site characteristics for use in SRP Section 3.8.4 to determine the normal and extreme winter precipitation loads on the roofs of seismic Category I structures. The normal winter precipitation roof load is a function of the normal winter precipitation event. The extreme winter precipitation roof loads are based on the weight of the antecedent snowpack resulting from the normal winter precipitation event plus the larger resultant weight from either: (1) the extreme frozen winter precipitation event, or (2) the extreme liquid winter precipitation event. Whereas the extreme frozen winter precipitation event is assumed to accumulate on the roof on top of the antecedent normal winter precipitation event, the extreme liquid winter precipitation event may or may not accumulate on the roof—that accumulation depends on the geometry of the roof and the type of drainage provided. DC/COL-ISG-7 further states:

- The normal winter precipitation event should be the highest ground-level weight (in lbf/ft$^2$) among: (1) the 100-year return period snowpack, (2) the historical maximum snowpack, (3) the 100-year return period two-day snowfall event, or (4) the historical maximum two-day snowfall event in the site region.

- The extreme frozen winter precipitation event should be the higher ground-level weight (in lbf/ft$^2$) between: (1) the 100-year return period two-day snowfall event, and (2) the historical maximum two-day snowfall event in the site region.

- The extreme liquid winter precipitation event is defined as the theoretically greatest depth of precipitation (in inches of water) for a 48-hour period that is physically possible over a 25.9-square-kilometer (10-square-mile) area at a particular geographical location during those months with the historically highest snowpacks.

In its response to RAI 02.03.01-14, dated December 18, 2008 (ML083570395), the applicant proposes a revision to FSAR Tier 2, Section 2.3S.1.3.4, which states that the ground level weight of the normal winter precipitation event and the weight of the extreme frozen winter precipitation event would both be 0.26 kPa (5.5 lbf/ft2). The staff found this revision acceptable because the value is the historic maximum snowfall in the site region and exceeds the calculated 100–year return period for a ground level snowpack value of zero lbf/ft2. The applicant also identifies the extreme liquid winter precipitation event to be 86.4 cm (34 in.), which is the same value previously identified by the applicant as the 48-hour PMWP site characteristic. As stated previously, the staff also found this value acceptable because the applicant determined the value in accordance with NUREG/CR–1486. The applicant has incorporated this information into Revision 3 of the FSAR. Therefore, RAI 02.03.01-14 is resolved and closed.

Both ABWR DCD Tier 1, Table 5.0 and Tier 2, Table 2.0-1, list a precipitation (for roof design) maximum snow load site parameter value of 2.39 kPa (50 lbf/ft$^2$). The combined ground level weight of the normal winter precipitation event and the extreme frozen winter precipitation event is 0.53 kPa (11.0 lbf/ft$^2$). As explained in Revision 4 of COL FSAR Tier 2, Subsection 3H.6.4.3.3.5, the applicant converts this ground load to a roof load of 0.63 kPa (13.2 lbf/ft$^2$), which is less than the roof maximum snow load site parameter value of 2.39 kPa (50 lbf/ft$^2$). The applicant also states in Revision 4 of COL FSAR Tier 2, Table 2.0-2, that the parapet height of the ABWR standard plant structures will be limited to 22.9 cm (9 in.). This amount of standing water is equivalent to a weight of approximately 2.25 kPa (47 lbf/ft$^2$). This limit should ensure that the roof maximum snow load site parameter value of 2.39 kPa (50 lbf/ft$^2$) will not be exceeded if an extreme liquid winter precipitation event occurs on an antecedent snowpack that has clogged the roof drains and scuppers. In Revision 4 of COL FSAR Tier 2, Subsection 3H.6.4.3.3.5, the applicant stated that because site-specific seismic Category I structures are designed without parapets, the roof load of the extreme liquid winter precipitation event cannot exceed the normal winter precipitation event and the extreme frozen winter precipitation event roof load of 0.63 kPa (13.2 lbf/ft$^2$) for these structures.

Because the ABWR precipitation (for roof design) maximum snow load standard plant site design parameter value bounds the corresponding STP site characteristics, COL License Information Item 2.1 is satisfied with respect to maximum snow load.

2.3S.1.4.3.7 Thunderstorms and Lightning

This discussion is intended to provide a general climatic understanding of the severe weather phenomena in the site region. However, the discussion does not generate site characteristics for use as design or operating bases.

The applicant estimates that, on average, there are approximately 56 days with thunderstorms per year in the site area. This frequency is taken from the NCDC local climatological annual summary data with comparative data for Victoria. The staff confirmed that the statistics provided by the applicant are correct.

Nearly 70 percent of these thunderstorms occur yearly between May and September. The applicant estimates approximately 7 flashes per square kilometer (17 flashes to earth per square mile) per year for the STP site area. The staff found this number appropriate based on similar values from NUREG/CR–3759, "Lightning Strike Density for the Contiguous United States from Thunderstorm Duration Records," an estimated mean annual ground flash density of 6 to 8 flashes per square kilometer (15 to 21 flashes per square mile), and the NWS Lightning Safety Web page[1] and a recorded flash density of 2 to 4 flashes per square kilometer (5 to 10 flashes per square mile) per year between the years 1996 and 2000.

2.3S.1.4.4 Meteorological Data for Evaluating the Ultimate Heat Sink

A description of the STP, Units 3 and 4, UHS is in FSAR Tier 2, Section 9.2.5, "Ultimate Heat Sink." The UHS is designed to provide sufficient cooling water to the reactor service water (RSW) system to permit a safe shutdown and cooling down of each unit and to maintain each unit in a safe shutdown condition. In the event of an accident, the UHS is designed to provide sufficient cooling water to the RSW system to safely dissipate the heat for the accident. The UHS is sized so that makeup water is not required for at least 30 days following an accident and design-basis temperature and chemistry limits for safety-related equipment are not exceeded. The UHS is designed to perform its safety function during periods of adverse site conditions, resulting in maximum water consumption and minimum cooling capability.

Each unit has its own UHS water storage basin. Above the basin is a counterflow mechanically induced draft cooling tower with six cooling tower cells. Two of these cells are dedicated to each of the three RSW divisions to remove heat from their respective reactor building cooling water (RCW)/RSW divisions. The RSW is pumped from the UHS water storage basin to the RCW heat exchangers for the removal of heat. The heated water is returned to the mechanically induced draft cooling tower where the heat is dissipated to the atmosphere by evaporation and conduction.

The UHS provides a source of cooling water that is available at all times for reactor operation, shutdown cooling, and accident mitigation. During normal plant operation, all three divisions are in operation with one cooling tower cell per division. When the heat load is increased during a cool down, shutdown, or accident, all cooling tower cells are in operation.

RG 1.27 specifies that applicants should ensure that design-basis temperatures of safety-related equipment are not exceeded and that a 30-day cooling supply is available. Consequently, applicants should identify the meteorological conditions that result in minimum water cooling as well as maximum 30-day evaporation and drift loss.

The applicant presents the results of the UHS thermal performance in FSAR Tier 2, Section 9.2.5.5. The applicant determines the worst-case meteorological conditions from a 45 year period (1961–2005) of sequential hourly wet-bulb, dry-bulb, and station atmospheric pressure data from Victoria. The applicant identifies the meteorological conditions resulting in minimum water cooling as a 1-day (24-hour) period occurring between September 16, 1996, and September 17, 1996, which resulted in the UHS basin water's maximum temperature. The applicant also identifies the meteorological conditions for maximum water usage as a 30-day (720-hour) period occurring between July 9, 1982, and August 7, 1982.

[1] NWS Lightning Safety Web page accessed on February 6, 2008, and is at: http:/www.lightningsafety.noaa.gov/lightning_map.htm.

The staff issued RAI 02.03.01-7, requesting the applicant to discuss the meteorological data used to evaluate the UHS performance. In particular, the staff was interested in the methodology used by the applicant to screen meteorological data in selecting the minimum water cooling and maximum water usage conditions for use in evaluating the UHS thermal performance.

In its response to RAI 02.03.01-7, dated May 29, 2008 (ML091490166), the applicant stated that the applicant reviewed the 45-year period (1961–2005) of sequential hourly wet-bulb, dry-bulb, and station atmospheric pressure data from Victoria to determine three sets of data (the highest average dry-bulb temperature, the highest average wet-bulb temperature, and the highest average evaporation potential, where evaporation potential was defined as the difference between the moisture content of saturated air at the dry-bulb temperature minus the actual moisture content of the air) for two time periods (a consecutive 30-day period and a 1-day period). The applicant then conducted a UHS thermal performance analysis using these three sets of data to determine the maximum 30-day evaporation and the maximum one-day basin water temperature. The applicant incorporated this information into Revision 3 of FSAR Tier 2, Subsection 2.3S.1.4.

The staff performed an independent evaluation of the applicant's analysis by reviewing the 1973 through 2005 Victoria data available in DS-3505 format from the NCDC Web site. The staff identified the highest 24-hour average wet-bulb temperature (e.g., worst one-day meteorological condition that maximizes water temperature) and the highest 720-hour average evaporation potential (e.g., the worst 30-day meteorological condition that maximizes water usage). Although the staff did identify different time periods containing the highest 24-hour average wet-bulb temperature and 720-hour average evaporation potential values, the staff's resulting highest wet-bulb temperature and evaporation potential values were similar to those of the applicant. Therefore, RAI 02.03.01-7 is resolved and closed.

The staff issued RAI 02.03.01-8a, requesting the applicant to justify not including meteorological data from the Palacios, Texas, Municipal Airport Weather Station in the selection of the minimum water cooling and maximum water usage conditions for evaluating the UHS thermal performance. In issuing this RAI, the staff pointed out that FSAR Tier 2, Subsection 2.3S.3.4.1.4, states that Palacios is considered to be representative of the STP site, and data collected at Palacios from the years 1997, through 2001, were used to predict cooling tower plume impacts resulting from the operation of the STP, Units 3 and 4, RSW mechanical draft cooling towers. The staff also noted that hourly data for the years 1988 through 2007 were available from the NCDC Web site.

In its response to RAI 2.3.1-8a, dated June 26, 2008 (ML081970231), the applicant stated that the UHS performance evaluation uses an 18-year period of data (years 1988 through 2005) from Palacios. The applicant stated that: (1) maximum water usage would be bounded by the results of the analysis using the Victoria data, and (2) maximum water temperature would be 0.3 °C (0.5 °F) higher than the results from the Victoria data but would still remain below the design limit cold water temperature of 35 °C (95 °F). The applicant summarizes the effects from using the Palacios data on the UHS performance in Revision 3 of COL FSAR Tier 2, Subsections 2.3S.1.4 and 9.2.5.5. By the applicant incorporating this information into the FSAR, RAI 02.03.01-8a is resolved and closed.

The staff concluded that the applicant has identified appropriate meteorological conditions for evaluating the UHS performance by examining long-term regional records (i.e., 45 years of

Victoria data and 18 years of Palacios data) and identifying meteorological conditions representing maximum evaporation and drift loss of water and minimum water cooling.

### 2.3S.1.4.5 Design-basis Dry- and Wet-Bulb Temperatures

ABWR DCD Tier 1, Table 5.0 and Tier 2, Table 2.0-1, list zero-percent exceedance (i.e., historical maximum limit) and 1-percent exceedance of dry-bulb and coincident and noncoincident wet-bulb temperatures as well as 99-percent exceedance and 100-percent exceedance (i.e., historical minimum limit) of dry-bulb temperatures as ambient design temperature site parameters.[1] Consequently, the applicant compiled zero-percent exceedance dry-bulb and coincident and noncoincident wet-bulb temperatures and 100-percent exceedance dry-bulb temperatures as STP, Units 3 and 4, ambient design temperature site characteristics based on data recorded for Victoria during the years 1971 through 2000. The applicant also identified one-percent exceedance dry-bulb and coincident and noncoincident wet-bulb temperatures and 99-percent exceedance dry-bulb temperatures as STP, Units 3 and 4, ambient design temperature site characteristics based on statistical data published by ASHRAE for Palacios Municipal Airport for the years 1987 through 2001. FSAR Tier 2, Table 2.0-2 presents both the ABWR DCD ambient design temperature site parameters and the corresponding STP, Units 3 and 4, ambient design temperature site characteristics chosen by the applicant.

Palacios is the closest climatic observation station to the STP, Units 3 and 4, site (located approximately 21 km [13 mi] to the west-southwest) with hourly temperature and humidity data. Because Palacios is located at approximately the same elevation as the STP, Units 3 and 4, site and is approximately the same distance from the Gulf of Mexico, the staff expects that the temperature and humidity data recorded at Palacios should be generally representative of STP, Units 3 and 4, site conditions. In order to confirm this hypothesis, the staff generated 1997, 1999, and 2000, Palacios dry-bulb statistics from the NCDC online database and compared them with similar statistics generated from the applicant’s 1997, 1999, and 2000, onsite meteorological database. The results of this comparison are as follows:

[1] The data presented in the ABWR DCD as minimum 1-percent exceedance and 0-percent exceedance values are also referred to by the staff as 99-percent exceedance and 100-percent exceedance values, respectively.

| DRY-BULB STATISTIC | 1997 | | 1999 | | 2000 | |
|---|---|---|---|---|---|---|
| | PALACIOS | STP | PALACIOS | STP | PALACIOS | STP |
| Maximum | 35.0 °C | 33.2 °C | 36.1 °C | 35.6 °C | 41.1 °C | 39.8 °C |
| 1% Exceedance | 32.8 °C | 31.3 °C | 32.8 °C | 32.1 °C | 33.9 °C | 32.5 °C |
| Median | 22.2 °C | 21.2 °C | 22.8 °C | 22.7 °C | 22.8 °C | 23.4 °C |
| 99% Exceedance | 2.2 °C | 1.4 °C | 3.9 °C | 4.3 °C | 2.8 °C | 3.3 °C |
| Minimum | -1.1 °C | -1.2 °C | 0.0 °C | 0.0 °C | -2.2 °C | 0.5 °C |
| °C=degrees Celsius. | | | | | | |

The staff also compiled and compared the 2007, hourly Palacios dew point statistics with the 2007, hourly onsite dew point data in the applicant's response to RAI 02.03.02-2:

| DEW POINT STATISTIC | 2007 | |
|---|---|---|
| | PALACIOS | STP |
| Maximum | 27.2 °C | 26.7 °C |
| 1% Exceedance | 26.1 °C | 25.4 °C |
| Median | 18.9 °C | 19.7 °C |

This comparison shows that the Palacios dry-bulb and dew point (humidity) data are generally representative (i.e., within 1 °C [1.8 °F]) of or slightly more conservative than the STP, Units 3 and 4, data.

The staff compared the applicant's 1-percent exceedance dry-bulb and coincident and noncoincident wet-bulb temperatures and 99-percent exceedance dry-bulb temperature STP, Units 3 and 4, ambient design temperature site characteristics with the Palacios data statistics published by ASHRAE. The staff confirmed that the statistics provided by the applicant are correct. The staff also calculated 100-year return period maximum and minimum dry-bulb and maximum wet-bulb statistics using 1988, to 2007, Palacios data and algorithms based on the Gumbel Type 1 extreme value distribution, as defined in Chapter 27 of the 2001, ASHRAE Handbook – "Fundamentals," for comparison with the Victoria zero-percent exceedance dry-bulb and noncoincident wet-bulb temperatures and 100-percent exceedance dry-bulb temperatures identified by the applicant as STP, Units 3 and 4, ambient design temperature site characteristics. The staff found that the Victoria zero-percent and 100-percent exceedance dry-bulb temperatures presented by the applicant bound the Palacios 100-year return maximum and minimum dry-bulb values calculated by the staff, but the Victoria 100-percent exceedance wet-bulb value identified by the applicant was approximately 1.4 °C (2.5 °F) lower (i.e., less conservative) than the Palacios 100-year return period maximum wet-bulb value calculated by the staff.

The applicant also generated 100-year return period maximum and minimum dry-bulb and maximum wet-bulb statistics through linear regression of individual daily maximum and minimum dry-bulb temperatures and daily maximum wet-bulb temperatures recorded during a 30-year period (years 1971 to 2000) at Victoria. The staff found that the Victoria 100-year return period maximum and minimum dry-bulb values calculated by the applicant (44 °C [111.3 °F] and -15.8 °C [3.6 °F], respectively) bound the Palacios 100-year return period maximum and minimum dry-bulb values calculated by the staff. The staff also found that the Victoria 100-year return period maximum wet-bulb value calculated by the applicant (30 °C [86.1 °F]) was within 0.5 °C (0.9 °F) of the Palacios 100-year return period maximum wet-bulb value calculated by the staff.

The applicant also presented 0.4-percent exceedance and two-percent exceedance dry-bulb and coincident and non-coincident wet-bulb temperatures and 99.6-percent exceedance and 100-percent exceedance dry-bulb temperatures based on 1987-2001, Palacios Municipal

Airport data published by ASHRAE. The staff compared the applicant's Palacios data against the published ASHRAE data to confirm that these statistics provided by the applicant are correct.

Because the Palacios 100-year return period maximum wet-bulb value appeared to exceed the Victoria 100-percent exceedance wet-bulb value, the staff issued RAI 02.03.01-8b, requesting the applicant to justify not including meteorological data from Palacios in the selection of zero-percent exceedance coincident and noncoincident wet-bulb temperatures and the 100-year return period maximum wet-bulb temperature ambient design temperature site characteristics discussed in FSAR Tier 2, Section 2.3S.1.5.

In its response to RAI 02.03.01-8b, dated June 26, 2008 (ML081970231), the applicant stated that the applicant analyzed twenty years (1988–2007) of hourly meteorological data collected at Palacios and found the following:

- A maximum recorded dry-bulb temperature of 41.1 °C (106 °F) with a coincident wet-bulb temperature of 25.4 °C (77.8 °F).
- A maximum recorded noncoincident wet-bulb temperature of 30.1 °C (86.1 °F).
- A 100-year return period maximum noncoincident wet-bulb temperature of 31.3 °C (88.3 °F).

Although the Palacios maximum recorded and 100-year return period noncoincident wet-bulb temperatures exceeded the corresponding Victoria wet-bulb temperatures, the applicant chose not to include the Palacios data in the FSAR because the exceedances were slight.

The staff subsequently issued RAI 0 2.03.01-22, requesting the applicant to: (1) revise the STP, Units 3 and 4, zero-percent exceedance maximum dry-bulb and concurrent wet-bulb ambient design temperature site characteristics to include the higher of either the maximum historic dry-bulb value or the maximum 100-year return period dry-bulb value for Victoria; (2) revise the STP, Units 3 and 4, zero-percent exceedance maximum wet-bulb ambient design temperature site characteristic to include the higher of either the maximum historic wet-bulb value or the 100-year return period wet-bulb value for Palacios; and (3) revise the STP, Units 3 and 4, zero-percent exceedance minimum dry-bulb ambient design temperature site characteristics to include the lower of either the minimum historic dry-bulb value or the minimum 100-year return period dry-bulb value for Victoria.

In RAI 02.03.01-22, the staff explained that 10 CFR 52.79(a) (1) (iii), states that COL applicants must identify the meteorological characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena historically reported for the site and surrounding area and with a sufficient margin for the limited accuracy, quantity, and period of time for which the historical data have been accumulated. In order to be compliant with 10 CFR 52.79(a)(1)(iii), the staff believes ambient design temperature site characteristics should be based on the higher of either the historic or 100-year return period values. The staff considered temperatures based on a 100-year return period as providing a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, as required by the regulation.

In its response to RAI 02.03.01-22, dated May 26, 2009 (ML091490166), the applicant emphasizes that the presentation of temperature data in Revision 0 to FSAR Tier 2, Section 2.3S.1.5, satisfies the requirements of 10 CFR 52.79(a)(1)(iii). The staff evaluated the response as summarized below:

- The applicant's RAI response states that because ABWR DCD Tier 1, Table 5.0 and Tier 2, Table 2.0-1 define zero-percent exceedance as a historical limit, there is no requirement in the ABWR DCD for the STP COL application to use 100-year return period temperatures as site characteristic values. However, 10 CFR 52.79(a)(1)(iii) states that the most severe temperatures reported for the site and surrounding area as historical limits shall include a sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated. The staff considers temperatures based on a 100-year return period to provide a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated as required by the regulation.

- The applicant's RAI response also states that the applicant used data from Victoria instead of Palacios to calculate the zero-percent exceedance noncoincident wet-bulb temperatures because the applicant believed regulatory guidance specifies the minimum requirements for the amount of historical data necessary to develop the required projections and the minimum required amount of historical data were not available for Palacios. However, the staff believes the 20 years of recent Palacios data should not be discounted just because the minimum required amount of historical data (e.g., 30 years), as specified by the applicant, is not available.

Subsequently, the staff has asked the applicant in RAI 02.03.01-23, to make the following changes to the FSAR:

a. Revise FSAR Tier 2, Section 2.3S.1.5 to include the Palacios maximum recorded and 100-year return period dry-bulb and wet-bulb temperature site characteristic values presented in the response to RAI 02.03.01-8b.

b. Revise FSAR Tier 2, Table 2.0-2 to include the zero-percent exceedance maximum dry-bulb ambient design temperature site characteristic value based on the higher of either the maximum recorded dry-bulb value or the maximum 100-year return period dry-bulb value for either Palacios or Victoria and provide an estimate of the concurrent wet-bulb value based on the resulting dry-bulb value.

c. Revise FSAR Tier 1, Table 5.0 and Tier 2, Table 2.0-2, to include the zero-percent exceedance maximum nonconcurrent wet-bulb ambient design temperature site characteristic value based on the higher of either the maximum recorded noncoincident wet-bulb value or the 100-year return period non-coincident wet-bulb value for either Palacios or Victoria.

d. Revise FSAR Tier 2, Table 2.0-2 to include the zero-percent exceedance minimum dry-bulb ambient design temperature site characteristic value based on the lower of either the minimum recorded dry-bulb value or the minimum 100-year return period dry-bulb value for either Palacios or Victoria.

The applicant agrees to implement the requested FSAR changes in its response to RAI 02.03.01-23, dated October 29, 2009 (ML093430299). The staff confirmed that COL FSAR Tier 2, includes the requested FSAR changes. Therefore, RAIs 02.03.01-8b, 02.03.01-22, and 02.03.01-23, are resolved and closed.

- STD DEP 5.0-1 Site Parameter

FSAR Tier 2, Table 2.0-2 shows that the ABWR DCD zero-percent exceedance noncoincident and the one-percent exceedance coincident and noncoincident wet-bulb temperatures do not bound the corresponding STP, Units 3 and 4, site characteristics. This finding is identified as Departure STP DEP T1 5.0-1 and is addressed in SER Section 9.4.6.

2.3S.1.4.6 Restrictive Dispersion Conditions

Based on NOAA Air Resources Laboratory "Air Stagnation Climatology for the United States (1948–1998)," (Wang and Angell, 1999), the applicant estimates that high-pressure stagnation conditions, usually accompanied by light and variable wind conditions, can be expected at the proposed STP, Units 3 and 4, site about 30 days per year or about six cases per year, with a mean duration of about 5 days for each case. Stagnation conditions usually occur from May through October, with the highest incidences recorded between July and September. This three-month period also coincides with the lowest monthly mean wind speeds during the year, as reported by the local climatological data summary for Victoria.

The applicant also notes that from a climatological standpoint, the lowest morning mixing heights occur in the autumn and are highest during the spring. Conversely, afternoon mixing heights reach a seasonal minimum in the winter and a maximum during the summer, which is expected because of more intense summer heating. The applicant presents mixing height data compiled from the USDA Forest Service Ventilation Climate Information System, which reports statistical mean monthly morning and afternoon mixing heights and wind speeds for the contiguous United States as a function of longitude and latitude.

The staff confirmed by the review of NOAA Air Resources Laboratory "Air Stagnation Climatology for the United States (1948–1998)," (Wang and Angell, 1999) and data compiled from the USDA Forest Service Ventilation Climate Information System that the information presented by the applicant regarding restrictive dispersion conditions is correct. Section 2.3S.2 of this SER discusses the proposed STP, Units 3 and 4, site air quality conditions for design and operating considerations. Sections 2.3S.4 and 2.3S.5, of this SER discuss atmospheric dispersion site characteristics used to evaluate short-term, post-accident airborne releases and long-term routine airborne releases, respectively.

2.3S.1.4.7 Climate Changes

As specified in NUREG–0800, the applicability of data used to discuss severe weather phenomena that may impact the proposed COL site during the expected period of reactor operation should be substantiated. Long-term environmental changes and changes to the region resulting from human or natural causes may affect the applicability of the historical data to describe the site's climate characteristics. The staff believes current climate trends should be analyzed for potential ongoing environmental changes.

The applicant analyzed normal temperature and rainfall trends during a 70-year period for successive 30-year intervals by decade for the climate division in which the STP site is located.

The applicant stated that the normal (i.e., 30-year average) temperature has increased only slightly (0.17 °C [0.3 °F]) during the last decade (i.e., the 1961, to 1990, normal temperature versus the 1970, to 2000, normal temperature) and the normal rainfall has trended upward by approximately 11.4 cm (4.5 in.) during these periods in the last two decades.

The U.S. Global Change Research Program (USGCRP) released a report to the President and Members of Congress in June 2009 titled, "Global Climate Change Impacts in the United States," (ML100580077). This report was produced by an advisory committee chartered under the Federal Advisory Committee Act. The report summarizes the science of climate change and the impacts of climate change on the United States.

The USGCRP report found that the average annual temperature of the Southeast (which includes the Texas coastline where the STP, Units 3 and 4, site is located) did not change significantly during the past century as a whole, but the annual average temperature has risen about 1.1 °C (2 °F) since 1970, with the greatest seasonal increase in temperature occurring during the winter months. Climate models predict continued warming in all seasons across the Southeast and an increase in the rate of warming throughout the end of the 21st century. Under a low heat-trapping gas emission scenario average temperatures along the Texas coastline are projected to rise 1.1–1.7 °C (2–3 °F) from a 1961-1979, baseline by mid-century (2040-2059), while a higher emissions scenario yields a 1.7–2.2 °C (3–4 °F) increase in average warming.

The USGCRP report also states that there is a 5 to 10 percent increase in observed annual average precipitation from 1958 to 2008 in the region in the proposed location of the STP, Units 3 and 4. Future changes in total precipitation are more difficult to project than changes in temperature. Model projections of future precipitation generally indicate that southern areas of the United States will become drier. Except for indications that the amount of rainfall from individual hurricanes will increase, climatic models provide divergent results for future precipitation for most of the Southeast.

The applicant stated that the occurrence of all tropical cyclones within a 185-km (100-nmi) radius of the STP site has been somewhat cyclical during the available period of record (1851-2006), with a peak occurring in the 1940s and a secondary peak in the 1880s.
The USGCRP reports that the force and frequency of Atlantic hurricanes have increased substantially in recent decades, but the number of North American mainland hurricanes reaching land does not appear to have increased in the past century. The USGCRP reports that likely changes in the future for the United States and surrounding coastal waters include more intense hurricanes with related increases in wind and rain, but not necessarily an increase in the number of storms that make landfall.

The applicant stated that the number of recorded tornado events has generally increased since detailed records were routinely kept, beginning around 1950. However, some of this increase is due to a growing population, greater public awareness and interest, and technological advances in detection. The USGCRP reaches the same conclusion. The USGCRP further states that there is no clear trend in the frequency or strength of tornadoes since the 1950s for the United States as a whole.

The USGCRP reports that the distribution by intensity of the strongest 10 percent of hail and wind reports has changed little and there is no evidence of an observed increase in the severity of such events. Climate models project future increases in the frequency of environmental conditions favorable to severe thunderstorms. But the inability to adequately model the small-scale conditions involved in thunderstorm development remains a limiting factor in projecting the future character of severe thunderstorms and other small-scale weather phenomena.

In conclusion, the staff acknowledges that long-term climatic change resulting from human or natural causes may introduce changes into the most severe natural phenomena reported for the site. However, no conclusive evidence or consensus of opinion is available on the rapidity or nature of such changes. There is a level of uncertainty in projecting future conditions because the assumptions regarding the future level of emissions of heat-trapping gases depends on projections of population, economic activity, and choice of energy technologies. If it becomes evident that long-term climatic change is influencing the most severe natural phenomena reported at the site, the COL holders have a continuing obligation to ensure that their plants stay within the licensing basis.

#### *2.3S.1.5 Post Combined License Activities*

There are no post COL activities related to this section.

#### *2.3S.1.6 Conclusion*

The staff reviewed the application and found that the applicant has presented and substantiated information to establish the regional meteorological characteristics. The staff's review confirmed that the applicant has established the meteorological characteristics at the site and in the surrounding area acceptable to meet the requirements of 10 CFR 100.20(c) (2) and 100.21(d), with respect to determining the acceptability of the site.

The staff finds that the applicant has considered the most severe natural phenomena historically reported for the site and surrounding area in establishing its site characteristics. Specifically, the staff accepted the methodologies used to analyze these natural phenomena and determine the severity of the weather phenomena reflected in these site characteristics. Because the applicant has correctly implemented these methodologies, as described above, the staff has determined that the applicant has considered these historical phenomena with margin sufficient for the limited accuracy, quantity, and period of time in which the data have been accumulated.

The staff finds that the identified site characteristics meet the requirements of 10 CFR 52.79(a)(1)(iii), with respect to identifying the most severe of the natural phenomena historically reported for the site and surrounding area; and with a sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

In addition, the staff compared the additional information in the application to the relevant NRC regulations and the guidance in Section 2.3.1 of NUREG–0800. The staff's review finds that the applicant has adequately addressed COL License Information Item 2.1 in accordance with Section 2.3.1 of NUREG–0800, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

### 2.3S.2 Local Meteorology

#### *2.3S.2.1 Introduction*

This section of the FSAR addresses the local (site) meteorological characteristics, assessments of the potential influence of the proposed plant and its facilities on local meteorological conditions, the impact of these modifications on plant design and operation, and a topographical description of the site and its environs.

### *2.3S.2.2 Summary of Application*

This site-specific supplement in the FSAR describes the following:

- Summaries of the local (site) meteorology in terms of airflow, (average wind direction and wind speed, wind direction persistence), atmospheric stability, temperature, atmospheric water vapor (e.g., wet-bulb temperature, dew point temperature, or relative humidity), precipitation, fog, atmospheric stability, and air quality;

- A topographical description of the site and its environs, as modified by the plant structures, including the site boundary, exclusion zone, and low population zone;

- An assessment of the construction and operation impacts of the plant and its facilities on the local meteorological parameters listed above; impacts include the effects of plant structures, terrain modification, and heat and moisture sources due to plant operation.

In addition, in FSAR Tier 2, Section 2.3S.2, the applicant provides the following:

*COL License Information Item*

- COL License Information Item 2.9 Local Meteorology

This site-specific supplement addresses COL License Information Item 2.9 from the certified ABWR DCD, which states that COL applicants will provide local meteorology for NRC review.

### *2.3S.2.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the local meteorology, and the associated acceptance criteria, are in Section 2.3.2 of NUREG--0800. In particular, the regulatory requirements are 10 CFR 52.79(a)(1)(iii), 10 CFR 100.20(c)(2) and 100.21(d).

The staff considered the following regulatory requirements in reviewing the applicant's discussion of the local meteorology:

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

- 10 CFR 100.20(c)(2) and 100.21(d), with respect to the consideration that has been given to the local meteorological and air quality characteristics of the site and other physical characteristics of the site that can influence the local meteorology.

NUREG–0800, Section 2.3.2, specifies that an application meets the above requirements if the application satisfies the following criteria:

- Provides local summaries of meteorological data that are based on onsite measurements in accordance with RG 1.23 and NWS station summaries (or other standard installation summaries) from appropriate nearby locations (e.g., within 80.5 km [50 mi]) and are presented as specified RG 1.206, Regulatory Position C.I.2.3.2.1.

- Provides a complete topographical description of the site and environs to a distance of 80.5 km (50 mi) from the plant, as described in RG 1.206, Regulatory Position C.I.2.3.2.2.

- Provides a discussion and evaluation of the influence of the plant and its facilities on the local meteorological and air quality conditions.

- Identifies potential changes in the normal and extreme values resulting from plant construction and operation.

- Provides a description of local site airflow that includes wind roses and annual joint frequency distributions of wind speed and wind direction by atmospheric stability for all measurement levels using the criteria in RG 1.23.

When independently assessing the veracity of the information presented by the applicant in FSAR Tier 2, Section 2.3S.2, the staff applied the same methodologies and techniques cited above.

### *2.3S.2.4 Technical Evaluation*

The staff reviewed the application and the applicant's responses to the RAIs to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding local meteorology. The staff followed the procedures in Section 2.3.2 of NUREG–-0800, as part of this review.

The staff reviewed the following information in the COL FSAR:

<u>*COL License Information Item*</u>

- COL license Information Item 2.9 Local Meteorology

The staff reviewed the site-specific information describing the local meteorology of the site and vicinity surrounding STP, Units 3 and 4. The staff's findings are presented below:

2.3S.2.4.1 Data Sources

The applicant used data from the existing STP, Units 1 and 2, meteorological monitoring program and 15 surrounding NWS observation stations listed in FSAR Tier 2, Table 2.3S-1 to describe local meteorology. The applicant used data from the onsite meteorological monitoring program to describe wind speed, wind direction, and atmospheric stability conditions. The applicant used data from surrounding offsite observation stations for temperature, atmospheric moisture, precipitation, and fog conditions.

2.3S.2.4.2 Normal, Mean, and Extreme Values of Meteorological Parameters

The applicant presents means and historical extremes of temperature, rainfall, and snowfall data in FSAR Tier 2, Tables 2.3S-3 "Climatological Extremes at Selected NWS and Cooperative Observing Stations in the STP 3 & 4 Site Area," and 2.3S-5, "Climatological Normals at Selected NWS and Cooperative Observing Stations in the STP 3 & 4 Site Area," from the 15 offsite observation stations listed in FSAR Tier 2, Section 2.3S.1. The staff evaluated the information submitted by the applicant for local meteorological conditions. The staff used data from the STP, Units 1 and 2, onsite meteorological monitoring system, as well as climatic data reported by the NCDC described below.

2.3S.2.4.2.1 Average Wind Direction and Wind Speed Conditions

The applicant provides hourly wind data from the STP, Units 1 and 2, onsite meteorological monitoring program, described in FSAR Tier 2, Section 2.3S.3, from the years 1997, 1999, and 2000. The applicant also presents monthly, seasonal, and annual wind roses based on 10-m and 60-m (33- and 197-ft) observation heights.

The staff confirmed that the wind directions from both levels are fairly similar. The prevailing annual wind direction for the site is generally from the south-southeast, with nearly 40 percent of the winds blowing from the southeast-through-south sectors. During the winter months, a bimodal direction distribution is exhibited with northerly winds (from the north-northwest through the north-northeast sectors) occurring with about the same frequency as winds from the southeast-through-south sectors. Winds from the southeast quadrant predominate during the spring and summer, with prevailing seasonal directions shifting from the southeast to the south as spring moves into summer. Autumn is predominated by winds from the southeast and northeast quadrants. The applicant reports that information from Victoria also indicates a prevailing south-southeasterly wind on an annual basis.

The applicant stated that annual average wind speeds at the 10- and 60-m (33- and 197-ft) observation levels are 4.1 m/s and 6.0 m/s (13.5 ft/s and 19.7 ft/s), respectively, which is generally consistent with the 6.1-m (20.0 ft) measurement height average wind speed of 4.3 m/s (14.1 ft/s) reported for Victoria for the years 1971 through 2000.

Palacios is the closest climatic observation station to the STP, Units 3 and 4, site (located approximately 21 km (13 mi) to the west-southwest), with hourly wind speed and direction data. Because of the proximity of Palacios to the proposed STP site and because of the similarity of topographic features at both locations (i.e., flat terrain and proximity to the Gulf of Mexico), the staff expects the wind data recorded at Palacios to be generally representative of STP, Units 3 and 4, site conditions.

In order to confirm this hypothesis, the staff generated a comparison of annual wind direction frequencies among the STP, Palacios, and Victoria hourly data for the years 1997, 1999, and 2000. This comparison, shown in SER Figure 2.3S.2-1, indicates a similar distribution among all three sites. The staff also compared annual average wind speeds among all three sites for the same three-year time period. The staff found that the STP 10-m (33-ft) level average wind speed of 4.1 m/s (13.5 ft/s) is consistent with the 6.1-m (20.0-ft) level average wind speed of 4.1 m/s (13.5 ft/s) at Victoria but somewhat lower than the 5.0 m/s (16.4 ft/s) average wind speed recorded at Palacios. The staff issued RAI 02.03.02-4a, requesting the applicant to justify not including meteorological data from Palacios in the review of average wind direction and wind speed conditions discussed in Revision 0 of FSAR Tier 2, Section 2.3S.2.2.1.

In its response to RAI 02.03.02-4a, dated June 26, 2008 (ML081970231), the applicant stated that the applicant evaluated a five-year period (years 1995 through 1999) of wind measurements from Palacios. Wind roses based on this data set showed reasonably similar characteristics in predominant directions on an annual basis, when compared to the annual onsite wind rose. The applicant also found that mean wind speeds at Palacios were similar, although somewhat higher, throughout the year when compared to the lower level wind speeds at the STP and Victoria. The applicant included this information on Palacios wind data in Revision 4 of COL FSAR Tier 2, Subsection 2.3S.2.2.1.

The staff agreed with the applicant that the winds for the proposed STP, Units 3 and 4, site are predominately from the southeast-through-south sectors. The staff also agreed with the applicant's documented annual average wind speeds of 4.1 m/s and 6.0 m/s (13.5 and 19.7 ft/s) at 10 and 60 m (33 and 197 ft). The staff's conclusions are based on a comparison between the STP onsite meteorological wind data and nearby hourly data reported at Palacios and Victoria. Therefore, RAI 02.03.02-4a is resolved and closed.

2.3S.2.4.2.2 Wind Direction Persistence

The applicant presents wind direction persistence and wind speed distribution summaries based on measurements at the STP site for the 3-year preoperational period of record (years 1997, 1999, and 2000). The summaries account for consecutive hours of wind direction from the same 22½-degree sector. The applicant reports in Revision 0 to FSAR Tier 2, Section 2.3S.2.2.2 that the longest persistence periods for each measurement height were 30 hours at the 10-m (33-ft) level (southeast sector) and 30 hours at the 60-m (197-ft) level (north and east-northeast sectors). The staff performed an independent analysis of these statistics and found two longer persistence periods at the 60-m (197-ft) level (a 32-hour period and a 33-hour period). The staff subsequently issued RAI 02.03.02-5, asking the applicant to confirm the length of the longest wind direction persistence period for the 60-m (197-ft) level and to revise the FSAR if necessary.

In its response to RAI 02.03.02-5, dated December 18, 2008 (ML083570395), the applicant explains that the hour listed in the FSAR wind direction persistence tables (FSAR Tier 2, Tables 2.3S-7 and 2.3S-8) is the lower limit within a period. In other words, the 30-hour frequency count identified in the persistence tables is for winds that persisted for at least 30 hours. The applicant clarifies this topic in Revision 3 of the FSAR. Therefore, RAI 02.03.02-5 is resolved and closed.

2.3S.2.4.2.3 Atmospheric Stability

The applicant classifies atmospheric stability in accordance with the guidance in RG 1.23. Atmospheric stability is a critical parameter for estimating dispersion characteristics in FSAR Tier 2, Sections 2.3S.4 and 2.3S.5. The dispersion of effluents is greatest for extremely unstable atmospheric conditions (i.e., Pasquill Stability Class A) and decreases progressively through extremely stable conditions (i.e., Pasquill Stability Class G). The applicant based the stability classification on temperature change with height (i.e., vertical temperature difference or delta-T) between the 60-m (197-ft) and 10-m (33-ft) height, as measured by the STP onsite meteorological monitoring program during the years 1997, 1999, and 2000.

The applicant provides seasonal and annual frequencies of atmospheric stability classes for the onsite preoperational three-year period of record. According to the applicant, there is a predominance of neutral stability (Pasquill Stability Class D) and slightly stable (Pasquill Stability Class E) conditions at the proposed STP site, which range from approximately 45 percent of the time during the autumn to approximately 63 percent of the time during the winter and spring. Extremely unstable conditions (Pasquill Stability Class A) occur most frequently during the summer and least frequently during the winter. Conditions that are extremely and moderately stable (Pasquill Stability Classes G and F, respectively) occur most frequently during the autumn and winter months.

The frequency of occurrence for each stability class is one of the inputs to the dispersion models used in FSAR Tier 2, Sections 2.3S.4 and 2.3S.5. The applicant includes these data in the form of a joint frequency distribution (JFD) of wind speed and direction data as a function of the stability class. A comparison of a JFD developed by the staff from the hourly data submitted by the applicant with the JFD developed by the applicant showed reasonable agreement.

Based on the stability data for the meteorological conditions at various US sites, a predominance of neutral (Pasquill Stability Class D) and slightly stable (Pasquill Stability Class E) conditions at the proposed STP site is generally consistent with expected meteorological conditions. A further discussion of the staff's review of the STP atmospheric stability data is in SER Subsection 2.3S.3.4.1.7.

2.3S.2.4.2.4 Temperature

The applicant characterizes normal and extreme temperatures for the site based on the 15 surrounding observation stations listed in FSAR Tier 2, Section 2.3S.2.1. The extreme maximum temperatures recorded near the site range from 39 to 44 °C (102 to 112 °F). The extreme minimum temperatures recorded near the site range from -15.6 to -10.6 °C (4 to 13 °F). Annual average temperatures for the 15 surrounding observation stations in the site vicinity, which are based on average daily mean maximum and minimum temperatures, range from 20.4 to 21.7 °C (68.8 to 71.1 °F). The applicant stated that the annual average diurnal (day-to-night) temperature differences in the site vicinity range from -11.4 to -5.7 °C (11.4 to 21.7 °F). In general, the greater diurnal temperature ranges occur at stations farther from the Gulf of Mexico and adjacent bays.

Using NCDC data, the staff reviewed the daily mean temperatures, extreme temperatures, and diurnal temperature ranges presented by the applicant. The staff issued RAI 02.03.01-17, requesting the applicant to confirm several of the extreme temperature statistics in FSAR Tier 2, Table 2.3S-3. Similarly, the staff issued RAI 02.03.01-20, requesting the applicant to confirm several of the mean temperature statistics in FSAR Tier 2, Table 2.3S-5. In its response to RAIs 02.03.01-17 and 02.03.01-20, dated December 18, 2008 (ML083570395), the staff revised several of the temperature statistics in FSAR Tier 2, Tables 2.3S-3 and 2.3S-5. The applicant includes these revised statistics in Revision 3 of the FSAR. Therefore, RAIs 02.03.01-17 and 02.03.01-20 are resolved and closed.

2.3S.2.4.2.5 Atmospheric Water Vapor

The applicant presents wet-bulb temperature, dew-point temperature, and relative humidity data summaries from the Victoria NWS observation station in Revision 0 to FSAR Tier 2, Subsection 2.3S.2.2.5 to characterize the typical atmospheric moisture conditions near the proposed STP site.

Based on 20 consecutive years of recorded data, the applicant indicates a mean annual wet-bulb temperature at Victoria of 18.1 °C (64.5 °F). The highest monthly mean wet-bulb temperature is 24.6 °C (76.2 °F) during July and the lowest is 10 °C (50.0 °F) during January. The applicant also indicates a mean annual dew-point temperature at Victoria of 16.1 °C (60.9 °F), which also reaches its maximum during summer and minimum during winter. The highest monthly mean dew-point temperature is 22.8 °C (73.1 °F) during July and August and the lowest is 7.8 °C (46.0 °F) during January.

Based on 30 consecutive years of recorded data, the applicant indicates that the annual relative humidity averages 76 percent at Victoria. The average early morning relative humidity levels exceed 90 percent from May through November, and they are not much lower during the remaining months of the year. Typically, the relative humidity values reach their diurnal maximum in the early morning and diurnal minimum during the early afternoon.

The staff verified the applicant's Victoria wet-bulb temperature, dew-point temperature, and relative humidity data by comparing the data with the NCDC "2006 Local Climatological Data, Annual Summary with Comparative Data, Victoria, Texas (KVCT)," (NOAA, 2007).

Palacios is the closest climatic observation station to the STP, Units 3 and 4, site with hourly temperature and humidity data. Because of the proximity of Palacios to the proposed STP site and because Palacios and the STP site are both located near warm bodies of water (Tres Palacios Bay and the main cooling reservoir, respectively), the staff expects the Palacios atmospheric moisture data to be typical of the atmospheric moisture conditions in the proposed STP site region. SER Subsection 2.3S.1.4.5, compares Palacios dew-point data with onsite dew-point data that support this conclusion. Therefore, the staff issued RAI 02.03.02-4b, asking the applicant to justify not including meteorological data from Palacios in the review of atmospheric water vapor discussed in FSAR Tier 2, Subsection 2.3S.2.2.5.

In its response to RAI 02.03.02-4b, dated June 26, 2008 (ML081970231), the applicant stated that it reviewed 20 years (1988 to 2007) of hourly Palacios data. The applicant found that: (1) the mean annual wet-bulb temperature at Palacios (19.1 °C [66.3 °F]) is higher than the Victoria temperature (18.1 °C [64.5 °F]); (2) the mean annual dew-point temperature at Palacios (17.3 °C [63.2 °F]) is higher than the Victoria temperature (16.1 °C [60.9 °F]); and (3) the annual average relative humidity at Palacios (80 percent) is higher than the 76 percent recorded at Victoria. However, the applicant did not provide a justification for not including this information in the FSAR. Consequently, as part of RAI 02.03.01-23, the staff asked the applicant to revise FSAR Tier 2, Subsection 2.3S.2.2.5 to include the Palacios wet-bulb, dew-point, and relative humidity data presented in the response to RAI 02.03.02-4b. In its response to RAI 02.03.01-23, dated October 29, 2009 (ML093430299), the applicant agreed to implement the requested FSAR changes.

The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed changes. Accordingly, the staff found that the applicant has adequately addressed this issue. Therefore, RAI 02.03.02-4b and RAI 02.03.01-23, are resolved and closed.

#### 2.3S.2.4.2.6 Precipitation

Based on data from the 15 surrounding observation stations, the applicant stated that the average annual precipitation (water equivalent) totals vary substantially (ranging from 88.3 to 145.4 cm [34.78 to 57.24 in.]). The applicant stated that the total annual rainfall tends to decrease more from east to west as a function of distance inland from the Gulf of Mexico and adjacent bay waters. The closest climatological stations to the STP site, which are all within 32 km (19.9 mi), have similar average rainfall totals ranging from 111.1 cm to 122 cm (43.75 in. to 48.03 in.). The applicant stated that the long-term average annual total rainfall at the STP, Units 3 and 4, site could reasonably be expected to be within this range.

According to the applicant, snowfall is rare. Normal annual totals range from a trace to 0.5 cm (0.2 in.). SER Subsection 2.3S.1.4.3.4, discusses snowfall in the vicinity of the proposed STP site in greater detail.

Using daily snowfall and rainfall data from NCDC, the staff independently verified the precipitation statistics in Revision 0 to FSAR Tier 2, Section 2.3S.2. The staff issued RAI 02.03.01-19, requesting the applicant to confirm several of the extreme snowfall historical statistics in FSAR Tier 2, Table 2.3S-3. Similarly, the staff issued RAI 02.03.01-20, requesting the applicant to confirm several of the mean snowfall statistics in FSAR Tier 2, Table 2.3S-5. In its response to RAIs 02.03.01-19 and 02.03.01-20, dated December 18, 2008 (ML083570395), the applicant revised several snowfall statistics in FSAR Tier 2, Tables 2.3S-3 and 2.3S-5. The applicant included these revised statistics in Revision 3 of the FSAR. Therefore, RAIs 02.03.01-19 and 02.03.01-20 are resolved and closed.

#### 2.3S.2.4.2.7 Fog

In Revision 0 to FSAR Tier 2, Section 2.3S.2.4.2.7, the applicant stated that Victoria is the closest station to the proposed STP site that makes fog observations. The applicant noted that, based on 43 consecutive years of recorded data, Victoria averages about 41.7 days per year of heavy fog conditions (e.g., visibility is reduced to 0.4 km [0.25 mi] or less). The peak frequency occurs during January, averaging approximately seven days per month. Heavy fog occurs least often during the summer, averaging less than one day per month during June, July, and August.

The staff confirmed the applicant's statement that the Victoria NWS station reports 41.7 days per year with heavy fog observations. However, Palacios is a closer climatic observation station to the STP, Units 3 and 4, site with hourly fog data. Because of the proximity of Palacios to the proposed STP site and because Palacios and the STP site are both located near bodies of water (Tres Palacios Bay and the main cooling reservoir, respectively), the staff expects the Palacios fog data to be typical of fog conditions in the proposed STP site region. Therefore, the staff issued RAI 02.03.02-4c, asking the applicant to explain not including meteorological data from Palacios in the review of fog data discussed in FSAR Tier 2, Subsection 2.3S.2.2.6.

In its response to RAI 02.03.02-4c, dated June 26, 2008 (ML081970231), the applicant stated that the record of fog data at Palacios is not as much or as complete as the data available from Victoria. Palacios started collecting fog data in late 2000, whereas the Victoria fog data reported by the applicant in FSAR Tier 2, Subsection 2.3S.2.2.7, covers 43 consecutive years of recorded data. The applicant reports an average annual frequency of about 29 days per year of heavy fog conditions at Palacios compared to an average of 42 days per year at Victoria. Nonetheless, the applicant still considers the frequency of heavy fog conditions at Victoria to be a reasonable indicator of the conditions that may be expected to occur at the STP site. Although the staff believes that the Palacios fog data are more representative of STP site conditions, the staff accepted the applicant's Victoria fog data because those data predict a higher (more conservative) frequency of heavy fog conditions. Therefore, RAI 02.03.02-4c is resolved and closed.

2.3S.2.4.3 Topographic Description

The proposed STP, Units 3 and 4, site is located in Matagorda County, Texas, approximately 19 km (12 mi) south-southwest of the city limits of Bay City, Texas. The applicant provides maps of topographical features within a 8-km and a 80.5-km (5-mi and a 50-mi) radius of the site. The applicant also provides terrain elevation profiles along each of the 16 standard 22½-degree compass radials to a distance of 80.5 km (50 mi). Based on these profiles, the applicant characterizes the proposed STP site terrain as basically flat to the northeast and southwest of the site, decreasing to sea level to the south toward the Gulf of Mexico and adjacent waters, and increasing gradually to the northwest to a maximum elevation of 50 m (164 ft) within 80.5 km (50 mi).

Based on topography data from the U.S. Geological Survey (USGS) and on a site visit, the staff agreed with this terrain characterization. The staff concluded that the applicant has provided the necessary topographic information.

2.3S.2.4.4 Potential Influence of the Plant and Related Facilities on Meteorology

The applicant stated that the associated paved, concrete, or other improved surfaces resulting from the construction of the proposed nuclear facility are insufficient to generate discernible, long-term effects to local or micro-scale meteorological conditions. Wind flow may be altered immediately adjacent to and downwind of larger site structures, but these effects will likely dissipate within 10 structure heights downwind. SER Section 2.3S.3, discusses the effects of these larger structures on wind flow.

The applicant stated that although temperature may increase above altered surfaces, the effects will be too limited in their vertical profile and horizontal extent to alter local- or regional-scale ambient temperature changes. Site clearing, grubbing, excavation, leveling, and landscape activities associated with plant construction will be localized and will not represent a significant change to the gently rolling topographic character of the site and its surrounding site area.

The staff agreed that the activities discussed above are too small-scale to impact the local meteorological characteristics of the site.

STP, Units 1 and 2, use the main cooling reservoir as a means of heat dissipation. Under normal operation, STP, Units 3 and 4, will also use the main cooling reservoir to dissipate waste heat rejected from the main condenser via the circulating water system (CWS). Mechanical draft cooling towers will also be used to remove heat load from the STP, Units 3 and 4, RSW system. The applicant stated in Revision 0 to FSAR Tier 2, Section 2.3S.2.4, that the potential meteorological effects due to the operation of the main cooling reservoir and these cooling towers may include enhanced ground-level fogging and icing, cloud shadowing and precipitation enhancement, and increased ground-level humidity.

The staff issued RAI 02.03.02-1, asking the applicant to describe the potential impacts of the main cooling reservoir and the RSW system mechanical draft cooling towers on the plant's design and operation. In particular, the staff asked the applicant to address the effects of local increases in ambient temperature, moisture content, and salt deposition on electrical transmission lines; electrical equipment (including transformers and switchyard); and heating, ventilation, and air conditioning (HVAC) intakes. In its response to RAI 02.03.02-1, dated May 29, 2008 (ML081560702), the applicant described the potential effects from increases in ambient temperature, moisture, and salt deposition on STP, Units 3 and 4, plant design and operation.

In its response to RAI 02.03.02-1, the applicant addressed the potential impacts from the main cooling reservoir. The applicant stated that salt deposition from the main cooling reservoir is not expected to affect HVAC systems and electrical equipment; most salt deposition resulting from the evaporation of main cooling reservoir water will remain in the pond. The additional water flow from STP, Units 3 and 4, to the main cooling reservoir will increase ambient moisture as a result of higher pond temperatures and evaporation. However, the applicant expects no adverse effects on plant features because HVAC intakes, transmission lines, and onsite electrical equipment are designed for outdoor operation, which includes environmental conditions such as fog and rain. Because the safety-related HVAC systems are designed for an outdoor summer temperature of 46.1 °C (115 °F), and the predicted maximum monthly main cooling reservoir discharge temperature for four-unit operation from the years 2003, to 2005, is 44.6 °C (112.3 °F) (from COL application Part 3, Table 3.4-3, “Environmental Report,”), the applicant stated that added heat from the main cooling reservoir is also not expected to adversely affect the HVAC systems. The staff concurred with these conclusions.

In its response to RAI 02.03.02-1, the applicant also addresses potential impacts on STP, Units 3 and 4, plant design and operation due to local increases in salt deposition and moisture from the RSW system using the Seasonal/Annual Cooling Tower Impact (SACTI) code. However, the applicant was in the process of modifying the UHS design at the time of this RAI response, and the revised design could impact the potential effects of the RSW system cooling towers on plant design and operation. Consequently, the staff issued RAI 02.03.02-7, requesting the applicant to update the information in the response to RAI 02.03.02-1, to reflect the revised UHS design. The staff also issued RAI 02.03.02-8, requesting the applicant to describe the assumptions and provide a copy of the SACTI input and output files that were used to estimate the fogging and drift impacts from the operation of the modified RSW system cooling towers.

In its response to RAI 02.03.02-7, dated April 14, 2009 (ML091070289), the applicant addressed potential impacts on the STP, Units 3 and 4, plant design and operation due to local increases in salt deposition and moisture from the modified RSW system using the SACTI code. The applicant stated that the maximum salt deposition rates at the bounding location for electrical equipment and transmission lines (i.e., the STP, Unit 4, transformers located approximately 380 m (1,247 ft) north-northwest of the UHS) will be between 1,100 and 4,200 kilograms per square kilometer (6,268 and 23,955 pounds per square mile) per month. The applicant stated that this amount represents a medium to heavy contamination environment, according to Institute of Electrical and Electronics Engineers (IEEE) Standard C57.19.100-1995, “IEEE Guide for Application of Power Apparatus Bushings,” reaffirmed December 9, 2003. However, the applicant also states these salt deposition rates are expected to be lower because they were calculated assuming the RSW system will be running at full capacity when in reality it is expected to run closer to half capacity. SER Section 8.2, addresses the countermeasures that will be taken by the applicant to prevent insulator and bushing failures on offsite power system equipment, as a result of salt deposits.

Because the SACTI model predicted no hours of fogging annually in any location, there should be little increase in ambient moisture operation affecting plant features. The applicant also states that added heat from the UHS is also not expected to adversely affect HVAC systems because the safety-related HVAC systems are designed for an outdoor summer temperature of 46.1 °C (115 °F), and the temperature of the exhaust plume from the UHS will not exceed the RSW return water temperature of 43.0 °C (109.4 °F).

In its response to RAI 02.03.02-8, dated April 14, 2009, the applicant stated that the cooling tower plume impacts were modeled with SACTI using 1997, 1999, and 2000 onsite wind speed, wind direction, and dry-bulb temperature data and concurrent total sky clearness, dew-point temperature, and ceiling height data from Palacios. FSAR Tier 2, Subsection 2.3S.3.2.1.2, states that relative humidity and temperature instrumentation were added to the 10-m and 60-m (33-ft and 197-ft) levels of the onsite meteorological tower in 2006, for the calculation of dew-point temperature to support estimates of the environmental impacts due to the operation of the STP, Units 3 and 4, RSW cooling towers. The staff issued RAI 02.03.03-2, asking the applicant to provide a copy of the onsite dew-point temperature database, once a contiguous year of data has been collected, and to compare these data to the Palacios dew-point data that were used to evaluate cooling tower plume impacts.

In its response to RAI 02.03.02-2, dated June 26, 2008 (ML081970231), provides the staff with a copy of a January 2007, through April 2008, hourly onsite dew-point temperature database. The applicant compared frequency distributions of the 2007, onsite data with 1997, 1999, and 2000, Palacios data and concluded that the Palacios data are generally consistent with the onsite dew-point temperature data. The staff performed an independent verification that compared the onsite 2007, dew-point temperature data with the 2007, Palacios dew-point temperature data and came to a similar conclusion that is explained in SER Subsection 2.3S.1.4.5. Consequently, the staff found the Palacios dew-point temperature data to be reasonably representative of onsite conditions and therefore acceptable for use in evaluating cooling tower plume impacts. For this reason, RAI 02.03.02-2 is resolved and closed.

The staff reviewed the SACTI computer code inputs and outputs provided by the applicant and concurred with the applicant’s analysis. Therefore, RAIs 02.03.02-1 and 02.03.02-8, are resolved and closed. The applicant revised Revision 3 to FSAR Tier 2, Subsection 2.3S.2.4, to describe the potential impacts of the main cooling reservoir and the RSW system mechanical draft cooling towers on plant design and operation discussed above. Therefore, RAI 02.03.02-7 is resolved and closed.

2.3S.2.4.5 Current and Projected Site Air Quality

The applicant stated in Revision 0 to FSAR Tier 2, Subsection 2.3S2.5.1, that the proposed STP, Units 3 and 4, site is located in the Metropolitan Houston-Galveston Intrastate Air Quality Control Region (AQCR 216). The applicant also notes that the counties within this region, including Matagorda County, have been designated as in attainment with or unclassified for all EPA air pollutant criteria (ozone, carbon monoxide, nitrogen dioxide, sulfur dioxide, particulate matter, and lead), except for a number of counties to the northeast or north-northeast of Matagorda County, which have been designated as “moderate” non-attainment with respect to the eight-hour ozone standard.

The staff issued RAI 02.03.02-6, asking the applicant to confirm the STP site's air quality status designations. In particular, the staff believed that: (1) the attainment status for AQCR 216 had not been designated for lead, and (2) the EPA had proposed to grant a request by the Governor of the State of Texas to voluntarily reclassify the AQCR 216 ozone nonattainment area from a moderate eight-hour ozone nonattainment area to a severe eight-hour ozone nonattainment area (72 *FR* 74252, December 31, 2007). In its response to RAI 02.03.02-6, dated December 18, 2008 (ML083570395), the applicant confirmed that: (1) the EPA granted a request from the Governor of the State of Texas to reclassify parts of AQCR-216 as a severe ozone nonattainment area to be effective October 31, 2008; and (2) the attainment status for lead has not been designated for most of the State of Texas. The applicant has incorporated this information into Revision 3 of the FSAR. Therefore, RAI 02.03.02-6 is resolved and closed.

According to the applicant, the proposed nuclear steam supply system (NSSS) and other radiological systems related to the proposed facility will not be sources of criteria pollutants or other air toxic emissions. Other proposed supporting equipment (e.g., emergency diesel generators, fire pump engines, combustion turbine) and other non-radiological emission-generating sources (e.g., storage tanks) or activities are not expected to be, in the aggregate, a significant source of criteria pollutant emissions.

Because the EPA has designated the proposed STP, Units 3 and 4, site area as in attainment or unclassified for all air pollutant criteria, and the new facility is not expected to be a significant source of air pollutants, the staff found that the STP, Units 3 and 4, site air quality conditions should not be a significant factor in the design and operating bases for the facility.

### *2.3S.2.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.3S.2.6 Conclusion*

The staff reviewed the application and found that the applicant has presented and substantiated information describing the local meteorological, air quality, and topographic characteristics important to evaluating the adequacy of the design and siting of this plant. The staff reviewed the information provided and, for the reasons given above, concludes that the identification and consideration of the meteorological, air quality, and topographical characteristics of the site and the surrounding area are acceptable and meet the requirements of 10 CFR 100.20(c)(2) and 10 CFR 100.21(d), with respect to determining the acceptability of the site.

The staff finds that the applicant has considered the appropriate site phenomena in establishing the site characteristics. Specifically, the staff has generally accepted the methodologies used to determine the meteorological, air quality, and topographic characteristics as documented in safety evaluation reports for previous licensing actions. Because the applicant has correctly implemented these methodologies, as described above, the staff has determined that the use of these methodologies results in site characteristics containing margin sufficient for the limited accuracy, quantity, and period of time in which the data have been accumulated.

The staff finds that the identified site characteristics meet the requirement of 10 CFR 52.79(a)(1)(iii), with respect to identifying the most severe of the natural phenomena historically reported for the site and surrounding area and with a sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

In addition, the staff compared the additional information in the application to the relevant NRC regulations and the guidance in Section 2.3.2, “Local Meteorology,” of NUREG–0800. The staff’s review concluded that the applicant has adequately addressed COL License Information Item 2.9, in accordance with Section 2.3.2 of NUREG–0800, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

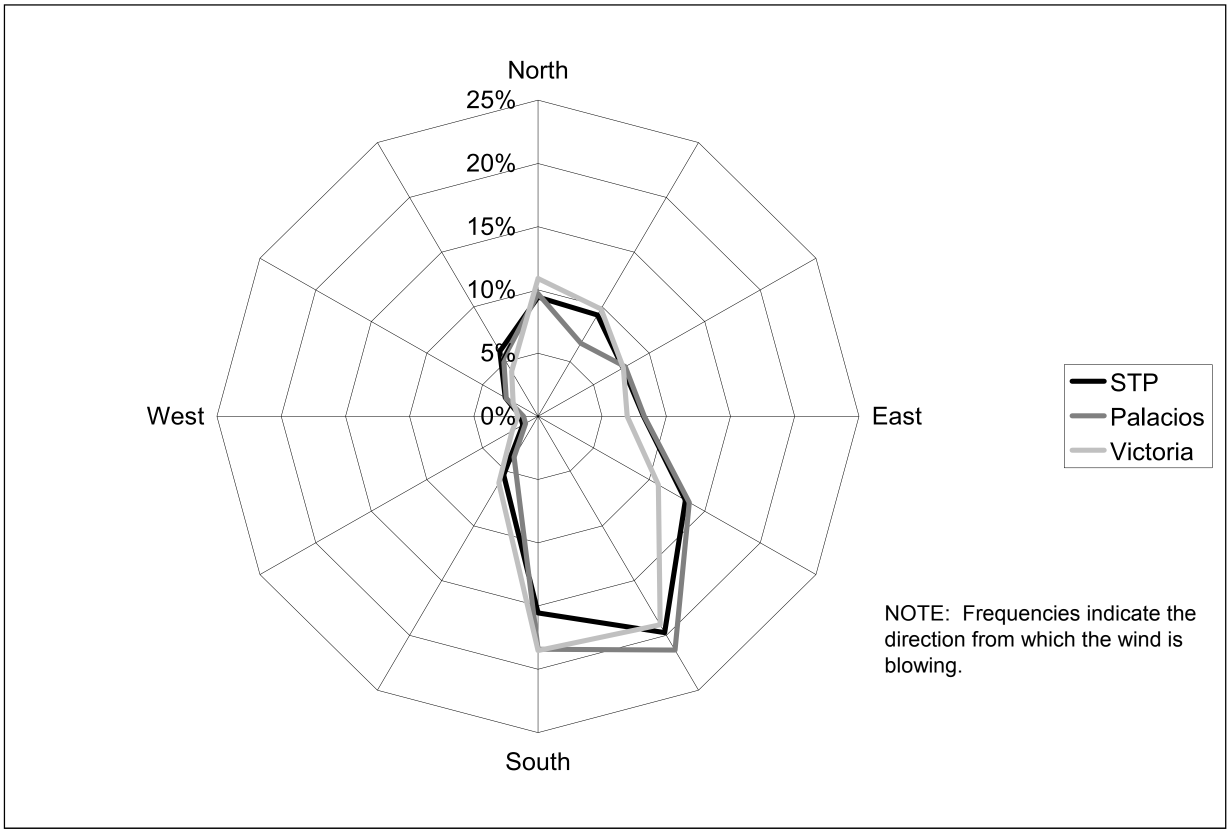


**Figure 2.3S.2-1 Comparison of 1997, 1999, and 2000 Wind Direction Frequency Distributions**

## 2.3S.3 Onsite Meteorological Measurements Program

### *2.3S.3.1 Introduction*

This section of the FSAR addresses the onsite meteorological monitoring program and the resulting data.

### *2.3S.3.2 Summary of Application*

This site-specific supplement included in the FSAR describes the following:

- A description of the pre-operational and operational meteorological monitoring program instrumentation, including the siting of sensors, sensor type and performance specifications, methods and equipment for recording sensor output, the quality assurance (QA) program for sensors and recorders, and data acquisition and reduction procedures

- The resulting meteorological database presented in the form of a joint frequency distribution of wind speed and direction by atmospheric stability class, and an hour-by-hour listing of the hourly-averaged parameters

In addition, in FSAR Section 2.3S.3, the applicant provides the following:

<u>*COL License Information Item*</u>

- COL License Information Item 2.10 Onsite Meteorological Measurements Program

This site-specific supplement addresses COL License Information Item 2.10, from the certified ABWR DCD, which states that COL applicants will provide a description of the onsite meteorological measurements program.

### *2.3S.3.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the onsite meteorological measurements program, and the associated acceptance criteria, are in Section 2.3.3, "Onsite Meteorological Measurements Programs," of NUREG–0800. In particular, the regulatory requirements are 10 CFR Part 20, "Standards for Protection Against Radiation," 10 CFR Part 50, 10 CFR 52.79, 10 CFR 100.20, "Factors to be Considered when Evaluating Sites," and 10 CFR 100.21, "Non-seismic siting criteria."

The staff considered the following regulatory requirements in reviewing the applicant's discussion of the site's location and description of the onsite meteorological measurements program:

- 10 CFR Part 20, Subpart D, "Radiation Dose Limits for Individual Members of the Public," with respect to the meteorological data used to demonstrate compliance with dose limits for individual members of the public.

- 10 CFR Part 50, Paragraphs 50.47(b)(4), 50.47(b)(8), and 50.47(b)(9), as well as Section IV.E.2 of Appendix E, "Emergency Planning and Preparedness for Production and Utilization Facilities," with respect to the onsite meteorological information available for determining the magnitude and continuously assessing the impact of the releases of radioactive materials into the environment during a radiological emergency.

- 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 19, "Control room," with respect to the meteorological data used to evaluate the personnel exposures inside the control room during radiological and airborne hazardous material accident conditions.

- 10 CFR Part 50, Appendix I, "Numerical Guides for Design Objectives and Limiting Conditions for Operation to Meet the Criterion "As Low As Reasonably Achievable" for Radioactive Material in Light-Water-Cooled Nuclear Power Reactor Effluents," with respect to meteorological data used in determining the compliance with numerical guides for design objectives and limiting conditions for operation to meet the requirement that radioactive material in effluents released to unrestricted areas be kept as low as is reasonable achievable (ALARA).

- 10 CFR 52.79(a)(1)(vi), with respect to a safety assessment of the site, including consideration of major SSCs of the facility and site meteorology, to evaluate the offsite radiological consequences at the EAB and LPZ.

- 10 CFR 100.20(c)(2), with respect to the meteorological characteristics of the site that are necessary for safety analysis or that may have an impact upon plant design in determining the acceptability of a site for a nuclear power plant.

- 10 CFR 100.21(c), with respect to the meteorological data used to evaluate site atmospheric dispersion characteristics and establish dispersion parameters such that: (1) radiological effluent release limits associated with normal operation can be met for any individual located off site, and (2) radiological dose consequences of postulated accidents meet prescribed dose limits at the EAB and LPZ.

NUREG–0800, Section 2.3.3, specifies that an application meets the above requirements if the application provides the following information:

- The pre-operational and operational monitoring program should be described, including: (1) a site map (drawn to scale) that shows tower location and true north with respect to man-made structures, topographic features, and other features that may influence site meteorological measurements, (2) distances to nearby obstructions of flow in each downwind sector, (3) measurements made, (4) elevations of measurements, (5) exposure of instruments, (6) instrument descriptions, (7) instrument performance specifications, (8) calibration and maintenance procedures and frequencies, (9) data output and recording systems, and (10) data processing, archiving, and analysis procedures.

- Meteorological data should be presented in the form of joint frequency distributions of wind speed and wind direction by atmospheric stability class, in the format described in RG 1.23. There should be an hour-by-hour listing of the hourly-averaged parameters in the format described in RG 1.23. If possible, evidence of how well these data represent long-term conditions at the site, possibly through a comparison with offsite data.

- At least two consecutive annual cycles (and preferably three or more whole years), including the most recent one-year period. The applicant should use these data to calculate: (1) the short-term atmospheric dispersion estimates for accident releases discussed in FSAR Tier 2, Section 2.3S.4, and (2) the long-term atmospheric dispersion estimates for routine releases discussed in FSAR Tier 2, Section 2.3S.5.

- The applicant should identify and explain any deviations from the guidance in RG 1.23.

When independently assessing the veracity of the information presented by the applicant in FSAR Tier 2, Section 2.3S.3, the staff applied the same methodologies and techniques cited above.

### *2.3S.3.4 Technical Evaluation*

The staff reviewed the application and the applicant's responses to the RAIs to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding the onsite meteorological measurements program. The staff followed the procedures described in Section 2.3.3 of NUREG–0800, as part of this review.

The staff reviewed the following information in the COL FSAR:

*COL License Information Item*

- COL License Information Item 2.10 Onsite Meteorological Measurements Program

The staff reviewed the preoperational and operational meteorological monitoring programs for STP, Units 3 and 4, including a description and site map showing tower locations with respect to manmade structures, topographic features, and other site features that can influence site meteorological measurements.

The staff's findings are summarized below.

2.3S.3.4.1 Preoperational Meteorological Measurement Program

The preoperational meteorological monitoring program was based on the pre-existing operational meteorological monitoring program and equipment used for STP, Units 1 and 2. The applicant stated that the STP, Units 1 and 2, Meteorological Monitoring Program are conducted in conformance with Revision 1 to RG 1.23.

2.3S.3.4.1.1 Tower Location

A 60-m (197-ft) guyed meteorological tower served as the primary data collection system and a 10-m (33-ft) freestanding tower served as a backup to the primary system. The backup meteorological system was a completely independent system installed and maintained for the purpose of providing redundant site-specific meteorological information at the 10-m (33-ft) level (i.e., wind speed, wind direction, temperature, wind direction standard deviation [sigma theta]). The primary tower was located approximately 2.1 km (1.3 mi) east of STP, Units 3 and 4, and the backup tower was located approximately 671 m (2,200 ft) south of the primary tower.

RG 1.23, states that to the extent practical, meteorological measurements should be made in locations that can provide data representative of the atmospheric conditions into which material will be released and transported. The tower or mast should be sited at approximately the same elevation as finished plant grade. Wind measurements should be made at locations and heights that avoid airflow modifications by obstructions such as large structures, trees, and nearby terrain. The sensors should be located over level, open terrain at a distance of at least ten times the height of any nearby obstruction, if the height of the obstruction exceeds one-half of the height of the wind measurement.

The applicant stated that both the primary and backup tower locations are clear of manmade and natural obstructions that could influence the collection of meteorological data. The bases of both the primary and backup towers were at an elevation of approximately 8.5 m (27.9 ft) mean sea level (MSL), while the finished plant grade of STP, Units 3 and 4, will range between 9.8 m (32.2 ft) and 10.4 m (34.1 ft) MSL. The terrain surrounding both meteorological towers is generally flat. Both towers are located in open fields with grassy surfaces, where the closest trees and brush range from 4.6 m to 9.2 m (15 ft to 30 ft) tall and are mostly 91 m (300 ft) or more from the towers. The nearby environmental shelters that house the processing and recording equipment are less than 5 m (16.4 ft) high, which is less than half of the lower level wind measurement height of 10 m (32.8 ft).

The applicant provides a map showing the locations of the meteorological towers with respect to the existing STP, Units 1 and 2, units. The tallest existing buildings are located more than 1.6 km (I mi) from the meteorological towers, so that the separation between these buildings and the meteorological towers is much greater than ten times the building heights.

The STP, Units 1 and 2, main cooling reservoir is approximately 1.6 km (1 mi) southwest of the primary meteorological tower. The potential impact of the main cooling reservoir on the primary tower measurements is discussed in SER Subsection 2.3S.3.4.1.7.

A staff visit to the STP meteorological towers during a pre-application site audit (between June 25, 2007, through June 26, 2007), confirmed the applicant's description of the general tower exposure.

The tower locations are consistent with the recommendations in RG 1.23. Therefore, the staff found the locations acceptable.

2.3S.3.4.1.2 Tower Design

The STP primary meteorological tower is a 60-m (197-ft), two level (60-m and 10-m [197-ft and 33-ft]) guyed triangular open lattice tower with a side width of approximately 111.8 cm (44 in.). Wind sensors are mounted on a boom extending 2.4 m (8 ft) outward on the upwind side of the tower to minimize tower structure influence.

The primary tower's open lattice design is consistent with the recommendations in RG 1.23. Therefore, the staff found that the design is acceptable.

2.3S.3.4.1.3 Instrumentation

The primary tower is instrumented with wind speed, wind direction, and ambient temperature at the 10-m and 60-m (33-ft and 197-ft) levels above ground level. Precipitation is measured at ground level near the base of the primary tower. Solar radiation is also monitored at 2.5 m (8.2 ft) above ground level. The vertical temperature difference (delta-T) is calculated as the difference between the temperatures measured at 60 m and 10 m (197 ft and 33 ft).
The dew-point temperature was also measured at the 3-m (10-ft) level. The applicant stated that additional relative humidity and temperature instrumentation were added to the 10-m and 60-m (33-ft and 197-ft) levels in 2006, for the calculation of the dew-point temperature to support estimates of the environmental impacts due to the operation of the STP, Units 3 and 4, RSW mechanical draft cooling towers.

RG 1.23, states that wind speed, wind direction, and vertical temperature differences should be measured at 10 and 60 m (33 ft and 197 ft) and at a third and higher level for stack releases of 85 m (279 ft) or higher. The highest release point is the 76-m (249-ft) reactor building plant stack. Therefore, the 10-m and 60-m (33-ft and 197-ft) measurement levels in the application are acceptable. The STP meteorological monitoring program also measures ambient temperature, precipitation, and atmospheric moisture, as specified in RG 1.23. Consequently, the applicant's primary tower meteorological parameters are consistent with the guidelines in RG 1.23.

The wind instrumentation consists of cup anemometers and wind vanes whose starting thresholds meet the 0.45 m/s (1.0 mph) criterion specified in RG 1.23. The ambient temperature sensors are platinum resistance temperature devices mounted in fan-aspirated radiation shields to minimize the impact of thermal radiation and precipitation. The rain gauge consists of a tipping bucket equipped with wind shields to minimize the loss of precipitation caused by the wind. The solar radiation sensor is a copper constantan thermopile. The relative humidity/temperature sensors, which were added in 2006 to determine the dew-point temperature, are capacitive polymer humidity and temperature sensors.

The applicant provides system performance specifications (e.g., system accuracy, measurement resolution) for the meteorological monitoring instrumentation that meet the criteria specified in RG 1.23. Because the instrumentation is consistent with the recommendations of RG 1.23, the staff found the instrumentation to be acceptable.

2.3S.3.4.1.4 Instrumentation Maintenance and Surveillance Schedules

RG 1.23 states that meteorological instruments should be inspected at a frequency that will ensure data recovery of at least 90 percent on an annual basis. Channel checks should be performed daily for operational monitoring programs and channel calibrations should be performed semiannually for both preoperational and operational programs, unless the operating history of the equipment indicates that either more or less frequent calibration is necessary.

The applicant stated that channel checks were performed daily and system calibrations were performed semiannually on both the primary and backup towers. Data recoverability for the 1997, 1999, and 2000 onsite meteorological database submitted in support of the STP, Units 3 and 4, COL application exceeded the RG 1.23 annual goal of 90 percent.

The instrument maintenance and surveillance schedules are consistent with the recommendations in RG 1.23. Therefore, the staff found that the schedules are acceptable.

2.3S.3.4.1.5 Data Reduction and Compilation

RG 1.23, states that meteorological monitoring systems should use electronic digital data acquisition systems as the primary data recording system. The digital sampling of data should be at least once every five seconds and the digital system should be compiled and archived as hourly values for use in historical climatic and dispersion analyses.

The applicant stated that independent microprocessors are used as the primary data collection system for the primary and backup meteorological towers, with digital data recorders used as a backup data collection system. The microprocessors sampling rate is once per second for each parameter, except for precipitation. Water collected by the rain gauge is automatically drained and counted each time an internal bucket filled with 0.25 mm (0.01 in.) of rainfall.
The microprocessors compile 15-minute and 60-minute data averages and compute sigma theta (wind direction standard deviation) data as well. The data are collected and electronically transmitted to various plant computers for data validation, screening, display, storage, and report generation. Computer programs are used in the screening process to identify recurring types of data errors, and the data are edited accordingly. Data reduction and compilation are consistent with the recommendations in RG 1.23 and are therefore acceptable to the staff.

2.3S.3.4.1.6 Deviations to Guidance from RG 1.23

The applicant did not report any deviations to the guidance in RG 1.23.

2.3S.3.4.1.7 Resulting Meteorological Data

The applicant provides joint frequency distributions of wind speed, wind direction, and atmospheric stability for both the 10-m and 60-m (33-ft and 197-ft) levels on the primary tower. The data are based on hourly measurements taken during the years 1997, 1999, and 2000. The applicant noted that the 1999 to 2000, 24-month period of data was determined to be the most defensible (i.e., using validated data with the least amount of data substitution); representative (i.e., tower and sensor siting in accordance with RG 1.23); and complete (i.e., annual data recovery rates in excess of 90 percent). There were no data older than ten years. Because RG 1.23 specifies that three or more years of data are preferable, the applicant also provides a third year—1997 data. The applicant provides a copy of the 1997, 1999, and 2000 hourly database to the staff.

The staff issued RAI 02.03.03-1, asking the applicant to describe in general terms any data substitution used to create the 1997, 1999, and 2000 onsite meteorological database. In its response to RAI 02.03.02-1, dated June 12, 2008 (ML081710126), the applicant noted that 204 hours of missing delta-T data are replaced with estimates using various techniques. Because these 204 hours represent a small fraction (less than one percent) of the total number of hours of delta-T data collected during 1997, 1999, and 2000, the staff found that the delta-T data substitution should not have a significant impact on the resulting onsite database. Consequently, RAI 02.03.02-1 is resolved and closed.

The staff performed a quality review of the 1997, 1999, and 2000 hourly meteorological database using the methodology described in NUREG–0917, “Nuclear Regulatory Commission Staff Computer Programs for Use with Meteorological Data,” dated July 1982. The staff used computer spreadsheets to further review the data. As expected, the staff’s examination of the data revealed generally stable and neutral atmospheric conditions at night and unstable and neutral conditions during the day. Wind speed and wind direction frequency distributions for each measurement channel were reasonably similar from year to year. The staff issued RAI 02.03.03-4, asking the applicant to explain the variation in onsite G Stability Class (extremely stable) frequency, which ranged from a maximum of 12.3 percent in 1999, to a minimum of 6.1 percent in 2000. In its response to RAI 02.03.03-4, dated June 12, 2008, the applicant stated that the applicant reviewed seven years of onsite stability class frequency distributions and found several similar year-to-year variations. Therefore, the applicant attributes the G stability class frequency differences to year-to-year variations, which are within the norm of the yearly variation. The staff found this response acceptable and RAI 02.03.03-4 is resolved and closed.

In a comparison between the lower and upper JFDs in FSAR Tier 2, Tables 2.3S-10, “Joint Frequency Distribution of Wind Speed and Wind Direction (10-Meter Level) by Atmospheric Stability Class for the STP 3 & 4 Site (1997, 1999, and 2000),” and 2.3S-11 “Joint Frequency Distribution of Wind Speed and Wind Direction (60-Meter Level) by Atmospheric Stability Class for the STP 3 & 4 Site (1997, 1999, and 2000),” and staff-generated JFDs from the hourly database provided by the applicant, the two sets of JFDs are similar. In order to show how well the 1997, 1999, and 2000 data set represents long-term conditions at the site, the staff compared the 1997, 1999, and 2000, 10-m (33-ft) wind direction; 10-m (33-ft) wind speed; and delta-T stability class frequency distributions with frequency distributions derived from the onsite data summaries in Section 2.3 of the STP, Units 1 and 2, Updated Final Safety Analysis Report (UFSAR). The STP, Units 1 and 2, UFSAR data summaries are based on data collected onsite from July 21, 1973, through July 20, 1976; and from October 1, 1976, through September 30, 1977. Although the two data sets are more than 30 years apart, there is a close correlation in wind direction with predominant winds from the south-southeast (see SER Figure 2.3S.3-1). There is also reasonable agreement in wind speed (see SER Figure 2.3S.3-2), with the median wind speed for the earlier data set slightly higher than the median wind speed for the later data set (4.3 m/s versus 3.8 m/s [14.1ft/s versus 12.5 ft/s]).

The stability class frequency distribution for both data sets in SER Figure 2.3S.3-3, also show reasonable agreement. Nonetheless, the staff issued RAI 02.03.03-3, asking the applicant to explain the six percent frequency increase of onsite Stability Class A (extremely unstable) conditions from the original data set to the current data set (see SER Figure 2.3S.3-3).

In its response to RAI 02.03.03-3, dated May 29, 2008 (ML081710126), the applicant stated that heat transfer from the main cooling reservoir will increase the lower level ambient temperature, enhance thermal instability, and result in more unstable atmospheric conditions. Commercial operation of STP, Units 1 and 2, commenced in August 1988, and June 1989, respectively. Therefore, the 1973 to 1977 data represent the STP, Units 1 and 2, pre-operational conditions; the 1997, 1999, and 2000, data represent the STP, Units 1 and 2, post-operational period. The main cooling reservoir is located approximately 1.6 km (1 mi) south-southwest of the main cooling reservoir. SER Figure 2.3S.3-4 shows that Stability Class A increased primarily in the south-through-southwest wind direction sectors for the STP, Units 1 and 2, post-operational period. Therefore, the applicant mainly attributes the six percent frequency increase of onsite Stability Class A to the thermal instability contributed by the main cooling reservoir. The staff found this assessment reasonable and RAI 02.03.03-3 is resolved and closed.

The staff subsequently issued RAI 02.03.03-5, stating the response to the RAI 02.03.03-3, assertion that an increase in measured onsite Stability Class A between the STP, Units 1 and 2, pre-operational period (years 1973–1977) and the STP, Units 1 and 2, post-operational period (years 1997, 1999, and 2000) is attributed to thermal instability contributed by the main cooling reservoir is in apparent conflict with the statement in Revision 0 to FSAR Tier 2, Subsection 2.3S.3.2.1.3. This subsection states that the influence of the main cooling reservoir on ambient temperature instrumentation is expected to be minimal due to the large separation in distance between the meteorological tower and the main cooling reservoir. In its response to RAI 02.03.03-5, dated November 20, 2008 (083290340), the applicant revised Subsection 2.3S.3.2.1.3 to address the impact on the meteorological tower by the main cooling reservoir in COL FSAR Revision 3. Therefore, RAI 02.03.03-5 is resolved and closed.

Based on: (1) an independent quality review of the onsite meteorological data, (2) a comparison with the onsite data summaries in the STP, Units 1 and 2, UFSAR, and (3) a comparison with offsite wind data in SER Subsection 2.3S.2.4.2.1, the staff accepted the three years of onsite data from the applicant as representative of the site. The staff also found the data to be an acceptable basis for estimating atmospheric dispersion for DBAs and routine releases in FSAR Tier 2, Sections 2.3S.4 and 2.3S.5.

The staff notes that the operation of STP, Units 3 and 4, could further increase the main cooling reservoir water temperatures. This increase would increase thermal instability, which could enhance the dispersion of releases occurring near the plant site beyond the prediction in FSAR Tier 2, Sections 2.3S.4 and 2.3S.5.

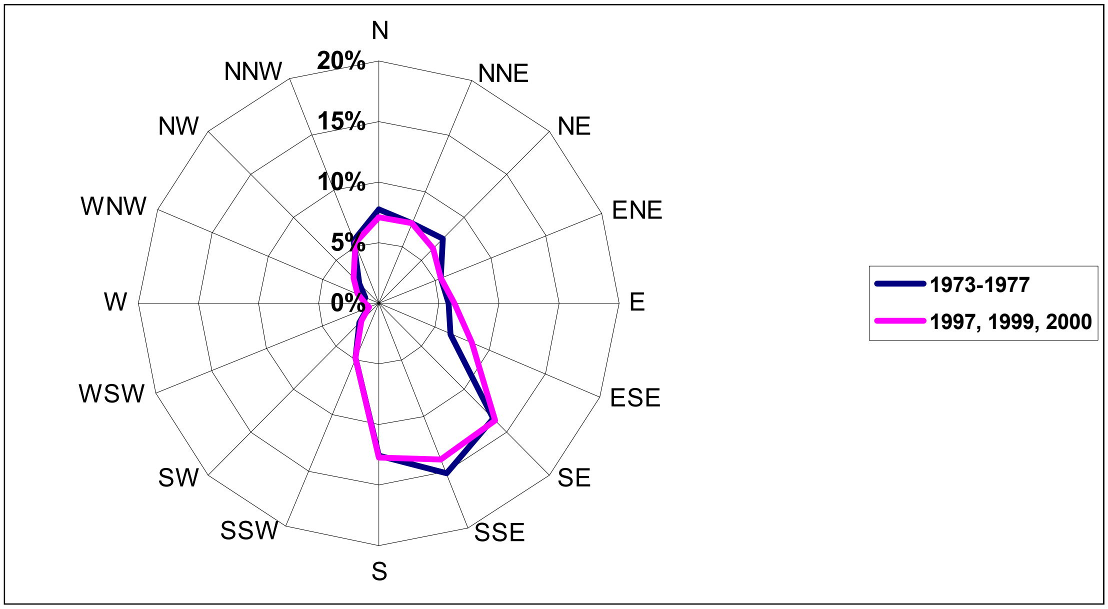


**Figure 2.3S.3-1 STP 10m Wind Direction Frequency Distributions**

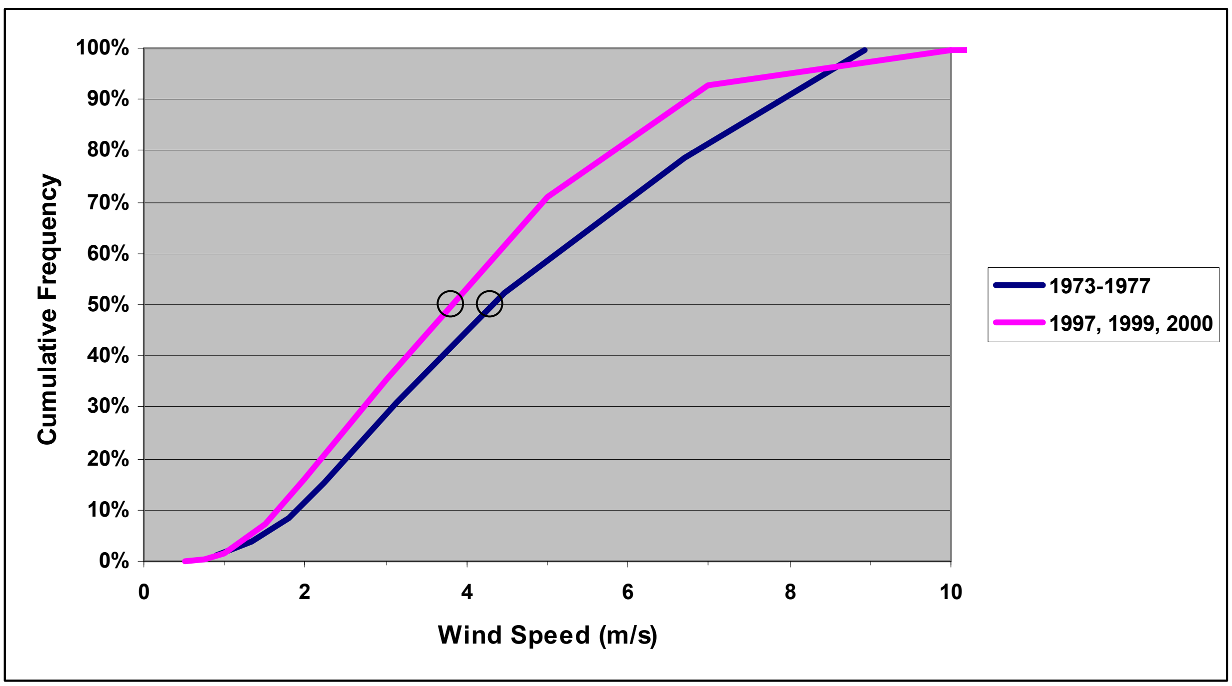


**Figure 2.3S.3-2 STP 10m Wind Speed Frequency Distributions**

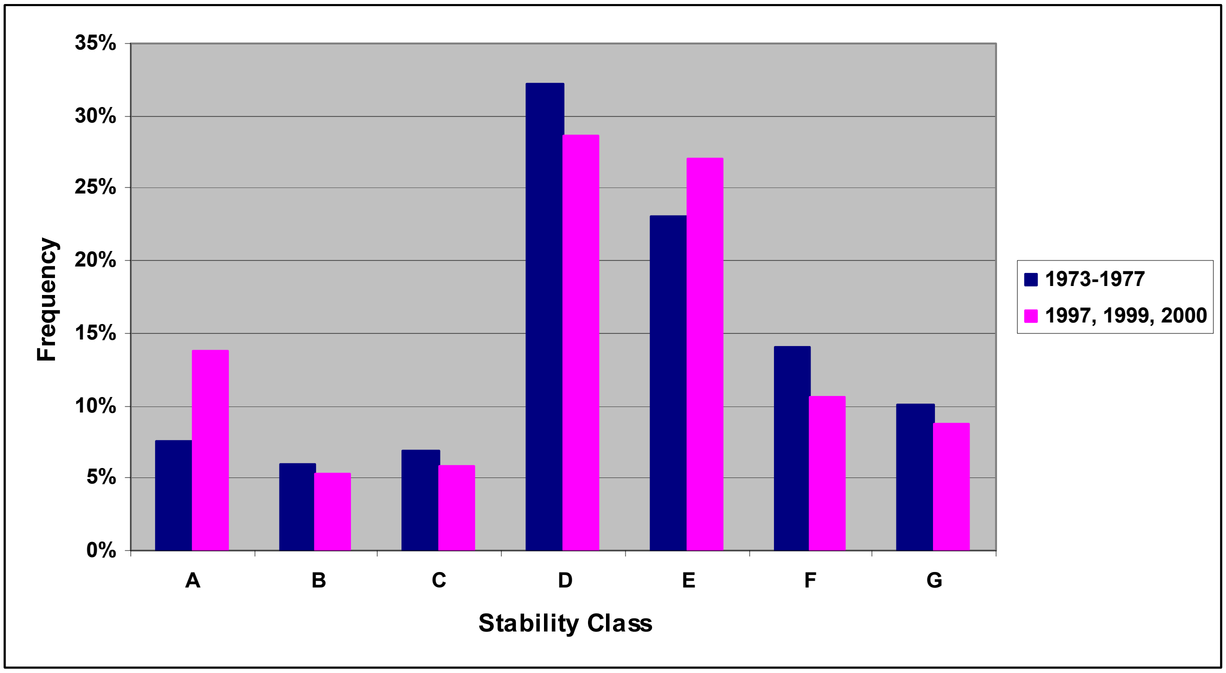


**Figure 2.3S.3-3 STP Stability Class Frequency Distributions**

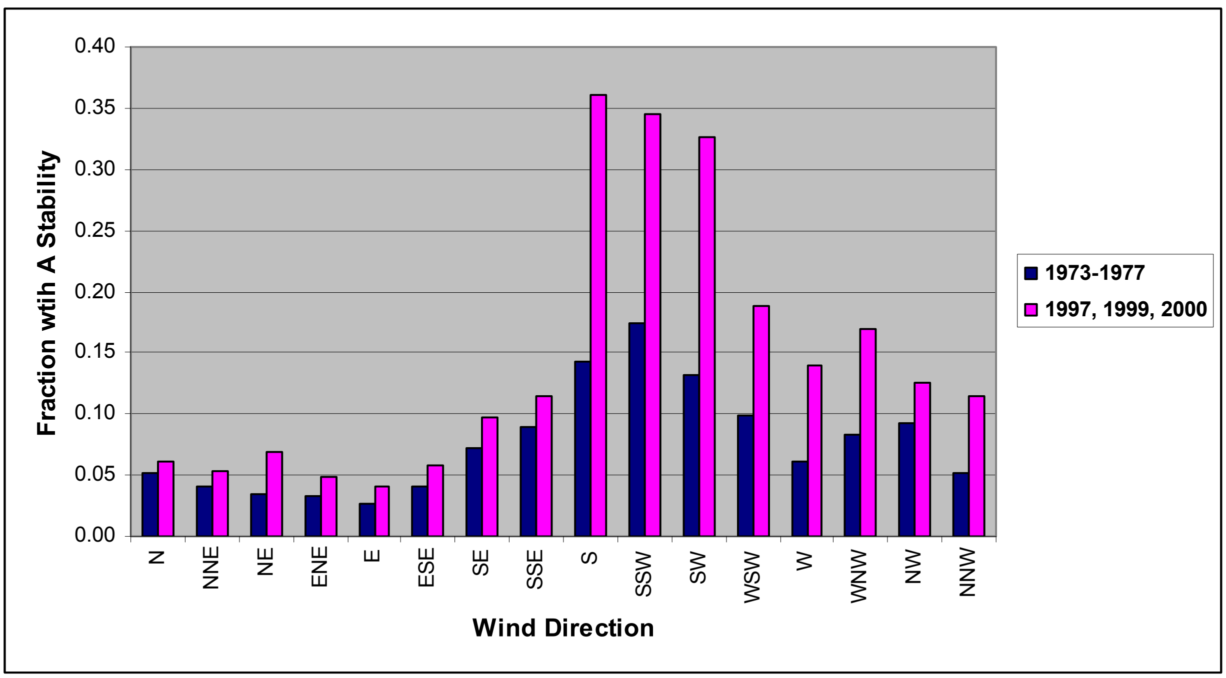


**Figure 2.3S.3-4 STP Stability Class A Frequency Distribution**

### 2.3S.3.4.2 Operational Meteorological Measurement Program

In Revision 0 to FSAR Tier 2, Subsection 2.3S.3.3, the applicant stated that the current meteorological system for STP, Units 1 and 2, will continue to be used during the operational phase for all four units. The staff issued RAI 02.03.03-6, asking the applicant to confirm whether the calibration and maintenance procedures described in FSAR Tier 2, Subsection 2.3S.3.2.3, and the data display, processing, archiving, and analysis in FSAR Tier 2, Subsection 2.3S.3.2.5, for the preoperational meteorological monitoring program will continue for the operational meteorological monitoring program. In its response to RAI 02.03.03-6, dated December 18, 2008 (ML083570395), the applicant revised Revision 3 to FSAR Tier 2, Subsection 2.3S.3.3. This revision affirms that the STP, Units 1 and 2, onsite meteorological monitoring program calibration and maintenance procedures described in FSAR Tier 2, Subsection 2.3S.3.2.3, and the data display, processing, archiving, and analysis procedures described in FSAR Tier 2, Subsection 2.3S.3.2.5, will continue to be used as the operational onsite meteorological monitoring program for STP, Units 3 and 4. Therefore, RAI 02.03.03-6 is resolved and closed.

The applicant provides a map showing the locations of the meteorological towers with respect to the existing STP, Units 1 and 2, units and the proposed STP, Units 3 and 4, units. The tallest existing and planned buildings for all four units are located more than 1.6 km (1 mi) from the meteorological towers, so that the separation between these buildings and the meteorological towers is much greater than 10 times the building heights. The proposed cooling system for STP, Units 3 and 4, includes the existing main cooling reservoir and two banks of mechanical draft cooling towers. The main cooling reservoir is approximately 1.6 km (1 mi) southwest of the primary meteorological tower and the cooling towers are located approximately 1.9 km (1.3 mi) west of both meteorological towers. The potential impact of the main cooling reservoir on the primary meteorological tower's delta-T measurements was discussed previously. The potential impact of the cooling towers on the primary meteorological tower's ambient temperature, dew-point, and relative humidity instrumentation is expected to be minimal because of the cooling tower's plume rise and because of the large separation distance between the cooling towers and the meteorological towers.

Since completing the collection of the preoperational meteorological data, the cup anemometers and wind vanes on both the primary and backup towers were replaced in 2005, with ultrasonic wind sensors. Some of the electronic microprocessors and data loggers have also been replaced. The system performance specifications (e.g., system accuracy and measurement resolution) continue to meet the criteria specified in RG 1.23. The data sampling rate continues to be once per second, and the data continue to be compiled into 15-minute and 60-minute averages. The 15-minute averaged data are compiled for real-time display in the STP, Units 3 and 4, control room, technical support center, and emergency operations facility. Emergency response dose assessments will be performed using the most recent 15-minute averaged data.

RG 1.23 states that provisions should be in place to obtain representative meteorological data (e.g., wind speed and direction representative of the 10-m (33-ft) level and an estimate of atmospheric stability) from alternative sources during an emergency, if the site meteorological program is unavailable. The backup tower measures wind speed, wind direction (including sigma theta for atmospheric stability class determination), and temperature at the 10-m (33-ft) level above ground level. Consequently, the backup tower meteorological parameters are also consistent with the guidelines in RG 1.23.

The applicant stated that provisions are currently in place to obtain representative regional meteorological data from the NWS or from a meteorological subcontractor during an emergency, if the site meteorological system becomes unavailable. The applicant also states that the current (or similar) emergency plan procedures and monitoring system arrangements will continue to be used for STP, Units 3 and 4. The proposed operational meteorological measurement program complies with the recommendations in RG 1.23. Therefore, the staff found the program to be acceptable.

### *2.3S.3.5 Post Combined License Activities*

Section 4 of Part 9, of the COL application contains emergency planning inspection, test, analysis, and acceptance criteria (EP-ITAAC). The following two EP-ITAAC involve demonstrating that the operational onsite meteorological monitoring program appropriately supports the STP, Units 3 and 4, emergency plan:

- EP-ITAAC 7.3: The means exists to continuously assess the impact of the release of radioactive materials into the environment, accounting for the relationship between effluent monitor readings and onsite and offsite exposures and contamination for various meteorological conditions. The acceptance criteria are: (1) the means exists to continuously assess the impact of the release of radioactive materials into the environment, accounting for the relationship between effluent monitor readings and onsite and offsite exposures and contamination for various meteorological conditions; and (2) the Emergency Plan Implementing Procedures and the Offsite Dose Calculation Manual calculate the relationship between effluent monitor readings and offsite exposure and contamination for various meteorological conditions.

- EP-ITAAC 7.4: The means exists to acquire and evaluate meteorological information. The acceptance criterion is that the means exists to acquire and evaluate meteorological information in that the following parameters are to be displayed in the technical support center and control room: wind speed (10 and 60 m [33 and 197 ft]), wind direction (10 and 60 m [33 and 197 ft]), vertical temperature difference (between 10 and 60 m [33 and 197 ft]), ambient temperature (10 m [33 ft]), and precipitation.

The EP and EP-ITAAC are addressed in SER Section 13.3, “Emergency Planning.”

### *2.3S.3.6 Conclusion*

The staff reviewed the information in Section 2.3S.3 of the COL FSAR and confirmed that the applicant has presented and substantiated information pertaining to the onsite meteorological monitoring program and the resulting database. The staff’s review found that the applicant has established the structure for the onsite meteorological monitoring program and the resulting database, which are acceptable and meet the requirements of 10 CFR 100.20, and 10 CFR 100.21, with respect to determining the acceptability of the site.

The staff finds that the onsite data also provide an acceptable basis for estimating atmospheric dispersion for DBA and routine releases from the plant. The data meet the requirements of General Design Criteria (GDC 19), ”Control Room,” 10 CFR 100.20, 10 CFR 100.21, 10 CFR Part 20, and Appendix I to 10 CFR Part 50. Finally, the equipment for measuring meteorological parameters during the course of accidents is sufficient to reasonably predict atmospheric dispersion of airborne radioactive materials, in accordance with 10 CFR 50.47(b) and Appendix E to 10 CFR Part 50.

In addition, the staff compared the additional information in the application to the relevant NRC regulations and the guidance in Section 2.3.3 of NUREG–0800. The staff's review finds that the applicant has adequately addressed COL License Information Item 2.10, in accordance with Section 2.3.3 of NUREG–0800, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

## 2.3S.4 Short-Term Atmospheric Dispersion Estimates for Accident Releases

### *2.3S.4.1 Introduction*

This section of the FSAR addresses the conservative atmospheric dispersion factor ($\chi$/Q or relative concentration) estimates at the EAB, the outer boundary of the LPZ, the control room, and technical support center (TSC) for postulated design-basis accidental radioactive airborne releases.

Dispersion estimates from the onsite and/or offsite airborne releases of hazardous materials, such as flammable vapor clouds, toxic chemicals, and smoke from fires, are reviewed in SER Section 2.2.3.

### *2.3S.4.2 Summary of Application*

This site-specific supplement in the FSAR describes the following:

- Atmospheric dispersion models to calculate atmospheric dispersion factors for postulated accidental radioactive airborne releases.
- Meteorological data and other assumptions used as inputs to atmospheric dispersion models.
- Derivation of diffusion parameters ($\sigma_y$ and $\sigma_z$).
- Determination of conservative $\chi$/Q values used to assess the consequences of postulated design-basis atmospheric radioactive releases to the EAB, LPZ, control room, and TSC.

In addition, in FSAR Tier 2, Section 2.3S.4, the applicant provides the following:

*COL License Information Items*

- COL License Information Item 2.1 Non-Seismic Design Parameters

This site-specific supplement addresses COL License Information Item 2.1, from the certified ABWR DCD, which states that "compliance with the envelope of standard plant site non-seismic design parameters of DCD Tier 2, Table 2.0-1 shall be demonstrated for design-bases events." DCD Tier 2, Section 2.2.1 further states that for design-basis events, the site is acceptable if all of the site characteristics fall within the envelope of ABWR standard plant site design parameters given in DCD Tier 2, Table 2.0-1. For cases where a characteristic exceeds its envelope, it will be necessary for the COL applicant to submit analyses to demonstrate that the overall set of site characteristics do not exceed the capability of the design. The DCD Tier 2, Table 2.0-1, envelope of ABWR standard plant site design parameters includes maximum 2-hour 95-percentile meteorological dispersion parameters for the EAB and maximum 2-hour 95-percentile and maximum annual average (8760 hours) meteorological dispersion parameters for the LPZ.

- COL License Information Item 2.11 Short-Term Atmospheric Diffusion Estimates for Accident Releases

This site-specific supplement addresses COL License Information Item 2.11, from the certified ABWR DCD, which states that COL applicants will provide site-specific, short-term dispersion estimates for the NRC's review to ensure that the envelope values of relative concentrations are not exceeded. Relative concentrations are located in the following tables from the ABWR DCD: Tables 15.6-3, "Instrument Line Break Accident Results"; 15.6-7, "Main Steamline Break Meteorology Parameters and Radiological Effects"; 15.6-13, "Loss of Coolant Accident Meteorology and Offsite Dose Results"; 15.6-14, "Loss of Coolant Accident Meteorology and Control Room Dose Results"; and 15.6-18, "Clean Up Water Line Break Meteorology and Dose Results."

### *2.3S.4.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the short-term atmospheric dispersion estimates for accident releases, and the associated acceptance criteria, are in Section 2.3.4 of NUREG–0800. In particular, the regulatory requirements are 10 CFR Part 50, 10 CFR 52.79 and 10 CFR 100.21.

The staff considered the following regulatory requirements in reviewing the applicant's discussion of short-term atmospheric dispersion estimates:

- 10 CFR Part 50, Appendix A, GDC 19, with respect to the meteorological considerations used to evaluate the personnel exposures inside the control room during radiological accident conditions.

- 10 CFR 52.79(a)(1)(vi), with respect to a safety assessment of the site, including consideration of major SSCs of the facility and site meteorology, to evaluate the offsite radiological consequences at the EAB and LPZ.

- 10 CFR 100.21(c)(2), with respect to the atmospheric dispersion characteristics used in the evaluation of EAB and LPZ radiological dose consequences for postulated accidents.

NUREG–0800, Section 2.3.4, specifies that an application meets the above requirements if the application provides the following information:

- A description of the atmospheric dispersion models used to calculate $\chi/Q$ values for accidental releases of radioactive and hazardous materials into the atmosphere.

- Meteorological data used for the evaluation (as inputs to the dispersion models), which represent annual cycles of hourly values of wind direction, wind speed, and atmospheric stability for each mode of accidental release.

- A discussion of atmospheric diffusion parameters, such as lateral and vertical plume spread ($\sigma_y$ and $\sigma_z$), as a function of distance, topography, and atmospheric conditions, should be related to measured meteorological data.

- Hourly cumulative frequency distributions of $\chi/Q$ values from the effluent release point(s) to the EAB and LPZ constructed to describe the probabilities that these $\chi/Q$ values will be exceeded.

- Atmospheric dispersion factors used for the assessment of consequences related to atmospheric radioactive releases to the control room for design-basis and other accidents.

- For control room habitability analysis, a site plan drawn to scale showing true North and potential atmospheric accident release pathways, control room intake, and unfiltered in leakage pathways.

In addition, the short-term atmospheric dispersion estimates for accident releases should be consistent with the appropriate sections from the following RGs:

- RG 1.23, provides criteria for an acceptable onsite meteorological measurements program; these data are used as inputs to atmospheric dispersion models.

- RG 1.145, Revision 1, “Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants,” presents criteria that characterize atmospheric dispersion conditions and evaluate the consequences of radiological releases to the EAB and LPZ.

- RG 1.194, “Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants,” presents criteria that characterize atmospheric dispersion conditions and evaluate the consequences of radiological releases to the control room.

When independently assessing the veracity of the information presented by the applicant in FSAR Tier 2, Section 2.3S.4, the staff applied the same methodologies, models, and techniques cited above.

### *2.3S.4.4 Technical Evaluation*

The staff reviewed the application and the applicant’s responses to RAIs to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding short-term atmospheric dispersion estimates for accident releases. The staff followed the procedures described in Section 2.3.4 of NUREG–0800 as part of this review.

The staff reviewed the following information in the COL FSAR:

<u>*COL License Information Items*</u>

- COL License Information Item 2.1 Non-Seismic Design Parameters

The staff’s review of the meteorological dispersion “Non-Seismic Design Parameters” is summarized below.

- COL License Information Item 2.11 Short-Term Atmospheric Diffusion Estimates for Accident Releases

The staff's review of the "Short-Term Atmospheric Dispersion Estimates for Accident Releases" is summarized below:

### 2.3S.4.4.1 Postulated Accidental Radioactive Releases

2.3S.4.4.1.1 Offsite Dispersion Estimates

a. Atmospheric Dispersion Model

The applicant uses the computer code PAVAN (NUREG/CR–2858, "PAVAN: An Atmospheric Dispersion Program for Evaluating Design-Basis Accidental Releases of Radioactive Materials from Nuclear Power Stations") to estimate χ/Q values at the EAB and at the outer boundary of the LPZ for potential accidental releases of radioactive material. The PAVAN model implements the methodology outlined in RG 1.145.

The PAVAN code estimates χ/Q values for various time-averaged periods ranging from 2 hours to 30 days. The meteorological input to PAVAN consists of a JFD of hourly values of wind speed and wind direction by atmospheric stability class. The χ/Q values calculated through PAVAN are based on the theoretical assumption that material released into the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the point of release and all distances for which χ/Q values are calculated.

For each of the 16 downwind direction sectors (N, NNE, NE, ENE, etc.), PAVAN calculates χ/Q values for each combination of wind speed and atmospheric stability at the appropriate downwind distance (i.e., the EAB and the outer boundary of the LPZ). The χ/Q values calculated for each sector are then placed in order from the greatest to the smallest, and an associated cumulative frequency distribution is derived based on the frequency distribution of wind speed and stabilities for each sector. The smallest χ/Q value in a distribution will have a corresponding cumulative frequency equal to the wind direction frequency for that particular sector. PAVAN determines for each sector an upper envelope curve based on the derived data (plotted as χ/Q versus probability of being exceeded), so that no plotted point is above the curve. From this upper envelope, the χ/Q value, which is equaled or exceeded 0.5 percent of the total time, is obtained. The maximum 0.5 percent χ/Q value from the 16 sectors becomes the 0–2 hour "maximum sector χ/Q value."

Using the same approach, PAVAN also combines all χ/Q values independent of wind direction into a cumulative frequency distribution for the entire site. An upper envelope curve is determined, and the program selects the χ/Q value that is equaled or exceeded no more than 5 percent of the total time. This value is known as the 0–2 hour "5-percent overall site χ/Q value."

The larger of the two χ/Q values, either the 0.5-percent maximum sector value or the 5-percent overall site value, is selected from the PAVAN output by the user to represent the χ/Q value for the zero–two hour time interval. Note that this resulting χ/Q value is based on one-hour averaged data, but it is conservatively assumed to apply for two hours.

To determine LPZ χ/Q values for longer time periods (e.g., 0–8 hours, 8–24 hours, 1-4 days, and 4–30 days), PAVAN performs a logarithmic interpolation between the 0-2 hour χ/Q values and the annual average (8,760 hours) χ/Q values for each of the 16 sectors and the overall site. For each time period, the highest among the 16-sector and overall site χ/Q values is identified and becomes the short-term site characteristic χ/Q value for that time period.

b. Meteorological Data Input

The meteorological input to PAVAN used by the applicant consisted of a JFD of wind speed, wind direction, and atmospheric stability based on hourly onsite data from the years 1997, 1999, and 2000. The wind data were obtained from the 10-m (33-ft) level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-T) measurements taken between the 60-m and 10-m (197-ft and 33-ft) levels on the onsite meteorological tower.

As discussed in SER Section 2.3S.3, the staff considers the 1997, 1999, and 2000 onsite meteorological database suitable for input to the PAVAN model.

c. Diffusion Parameters

The applicant chooses to implement the diffusion parameter assumptions, outlined in RG 1.145 as a function of atmospheric stability, for the PAVAN model runs. Both the EAB and outer boundary of the LPZ extend over the main cooling reservoir in the southeast clockwise to the south-southwest sectors. The staff consequently issued RAI 02.03.04-4, asking the applicant to describe the impact of reduced surface roughness resulting from over-water trajectories on the resulting short-term offsite atmospheric dispersion estimates. In its response to RAI 02.03.04-4, dated November 20, 2008 (ML083290340), the applicant stated that the reduced surface roughness induced by the main cooling reservoir will result in less mechanical turbulence and higher χ/Q values for those portions of the EAB and LPZ that extend over the main cooling reservoir. The applicant also states that reduced surface roughness will also increase ambient wind speed slightly and will subsequently minimize the net effect of reduced surface roughness on the offsite short-term atmospheric dispersion estimates. The staff also believes that low-level turbulent vertical mixing may be enhanced due to the warm water temperatures in the main cooling reservoir, which would also counteract the reduced surface roughness from over-water trajectories. Therefore, the staff concludes the applicant's use of diffusion parameter assumptions, as outlined in RG 1.145, acceptable and RAI 02.03.04-4 is resolved and closed.

d. Resulting Relative Concentrations

The applicant modeled one ground-level release point and did not take credit for building wake effects. Ignoring building wake effects for a ground-level release decreases the amount of atmospheric turbulence assumed to be in the vicinity of the release point, resulting in higher (i.e., more conservative) χ/Q values for a flat terrain site such as STP, Units 3 and 4. A ground-level release assumption that does not take credit for building wake effects is therefore acceptable to the staff.

FSAR Tier 2, Section 2.1S.2 states that the STP, Units 3 and 4, facilities are located within the EAB and LPZ already designated for STP, Units 1 and 2. The EAB is an oval with a minimum distance of approximately 1,430 m (4,692 ft) from the center of each of the STP, Units 1 and 2, reactor containment buildings. The center of the exclusion area "oval" is a point approximately 93 m (305 ft) directly west of the center of the STP, Unit 2, reactor containment building. This point is also the center of the existing LPZ, which is a circle with a radius of 4.8 km (3 mi). Because the distances to the EAB and LPZ from STP, Units 3 and 4, are different for each directional sector, the applicant stated in Revision 0 to FSAR Tier 2, Section 2.3S.4.2, that the shortest distances in each direction were used.

Regulatory Position C.1.2 of RG 1.145, states that for each of the 16 direction sectors, the distances to the EAB and LPZ to be used in the χ/Q calculations should be the minimum distance from the stack or, in the case of releases through vents or building penetrations, the nearest point on the building to the EAB or LPZ, within a 45-degree sector centered on the compass direction of interest. The staff issued RAI 02.03.04-3, asking the applicant to confirm that this approach was also used to derive the distances to the EAB and LPZ used in the χ/Q calculations.

In its response to RAI 02.03.04-3, dated June 12, 2008 (ML081710126), the applicant stated the releases to the EAB and LPZ are assumed to be located at the center of either STP, Unit 3 or 4, and not at the nearest point on the building complex to the EAB or LPZ, as discussed in RG 1.145. The applicant's response further states that the shortest distance to the EAB is in the northwest direction (930 meters [3,051 ft]), and the difference in the distance from the edge of the reactor building to the EAB is approximately 41 m (134.5 ft) shorter than if measured from the center of STP, Unit 4. The applicant believes this 41-meter (134.5-ft) difference in distance does not significantly affect the χ/Q values predicted at the EAB.

FSAR Tier 2, Figure 2.3S-23, "Accident Release and Receptor Locations," shows the assumed DBA release locations. One of these locations is the turbine building truck door, which is located at the NW corner of the turbine building. The staff estimated that this release location is approximately 120 m (394 ft) closer to the EAB in the northwest direction compared to the northwest edge of the reactor building. Therefore, the applicant's alternative approach to calculating downwind distances to the EAB and LPZ using the shortest distance from the center of either STP, Unit 3 or 4, as described in the response to RAI 02.03.04-3, was not convincing evidence that the calculation is conservative. The staff consequently issued RAI 02.03.04-5.

In its response to RAI 02.03.04-5, dated January 28, 2009 (ML090300648), the applicant revised the approach for calculating distances to the EAB and LPZ by defining a power block envelope that encloses the STP, Units 3 and 4, reactor buildings and turbine buildings. The applicant then determines the shortest distances from the power block envelope to the EAB and LPZ within 45-degree sectors centered on the compass directions of interest, in accordance with RG 1.145. Note that the revised set of distances showed that the shortest distance to the EAB was reduced from 930 m to 695 m (3,051ft to 2,280 ft) in the northwest direction. The applicant uses these revised distances in the PAVAN analysis to update the EAB and LPZ χ/Q values. The applicant proposes revisions to FSAR Tier 2, Section 2.3S.4 to reflect this revised approach for calculating distances to the EAB and LPZ. The staff found that the applicant's revised approach for calculating distances to the EAB and LPZ is consistent with the guidance of RG 1.145. This approach is therefore acceptable and RAI 02.03.04-3 is resolved and closed.

The applicant provided a revised response to RAI 02.03.04-5, dated July 30, 2009 (ML092150966). The response states that the EAB and LPZ χ/Q values are being revised using a slightly larger power block footprint. The applicant incorporates its revised set of EAB and LPZ χ/Q values into Revision 3 of the FSAR. The staff confirmed these revised atmospheric dispersion estimates by running the PAVAN computer model using information in the FSAR and the applicant's response to RAI 02.03.04-5, and obtained similar results. The staff found that the applicant's results bounded the staff's values. The staff therefore accepted the revised short-term EAB and LPZ χ/Q values presented by the applicant and RAI 02.03.04-5 is resolved and closed.

In accordance with COL License Information Item 2.1, FSAR Tier 2, Table 2.0-2 compares the site-specific EAB and LPZ χ/Q values with the ABWR standard plant meteorological dispersion site design parameters, which are listed in DCD Tier 2, Table 2.0-1. Note that FSAR Tier 2, Table 2.0-2 compares the PAVAN-generated, 0–2 hour 0.5 percent maximum sector χ/Q values and the 5 percent overall site χ/Q values with the ABWR DCD 0–2 hour 95 percentile meteorological dispersion site design parameters, which are χ/Q values that are expected to be exceeded no more than 5 percent of the time.

FSAR Tier 2, Table 2.0-2 concludes that the ABWR DCD EAB and LPZ χ/Q values are not exceeded. Smaller χ/Q values are associated with a greater dilution capability, resulting in lower radiological doses. When comparing a DCD site parameter χ/Q value and a site characteristic χ/Q value, the site is acceptable for the design if the site characteristic χ/Q value is smaller than the site parameter χ/Q value. Such a comparison shows that the site has better dispersion characteristics than the reactor design requires. The staff noticed in the applicant's response to RAI 02.03.04-5 that the revised PAVAN-predicted maximum 0–2 hour EAB and LPZ χ/Q values (2.74E-04 s/$m^3$ and 5.27E-05 s/$m^3$, respectively), which are in the proposed revision to FSAR Tier 2, Subsection 2.3S.4.2.1.1, differ from those χ/Q values listed in the proposed revision to FSAR Tier 2, Table 2.0-2 as STP, Units 3 and 4, site characteristic values (1.62E-04 s/$m^3$ and 3.99E-05 s/$m^3$, respectively) for comparison to the ABWR DCD EAB and LPZ χ/Q values. The staff issued RAI 02.03.04-9, requesting the applicant to explain this apparent discrepancy and update the FSAR if necessary.

In its response to RAI 02.03.04-9, dated October 29, 2009 (ML093430299), the applicant explained that the ABWR maximum 2-hour 95-percentile EAB χ/Q site parameter value of 1.37E-03 s/$m^3$ is compared in FSAR Tier 2, Table 2.0-2 to the PAVAN-generated 5-percent overall site EAB χ/Q site characteristic value of 1.62E-04 s/$m^3$; likewise, the ABWR maximum 2-hour 95-percentile LPZ χ/Q site parameter value of 4.11E-04 s/$m^3$ is compared to the PAVAN-generated 5-percent overall site LPZ χ/Q site characteristic value of 3.99E-05 s/$m^3$. The applicant adds that FSAR Tier 2, Table 2.0-2 will be updated to list both the PAVAN 0-2 hour, 0.5 percent maximum sector EAB and LPZ values and the PAVAN 0-2 hour, 5 percent overall site EAB and LPZ values. The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed changes. Therefore, RAI 02.03.04-9 is resolved and closed.

As a result of its review of the applicant's response to RAI 02.03.04-9, the staff issued RAI 02.03.04-11, requesting that the applicant revise its proposed footnotes to FSAR Tier 2, Table 2.0-2 to more accurately label the 0.5 percent maximum sector χ/Q values and the 5.0 percent overall site χ/Q values being presented in the table. In its response to RAI 02.03.04-11, dated March 1, 2010 (ML100620824), the applicant explained that it will update the proposed footnotes as requested by the staff. The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed footnote, per the applicant's commitment in the RAI response. Therefore, RAI 02.03.04-11 is resolved and closed.

In accordance with COL License Information Item 2.11, FSAR Tier 2, Table 15.6.5S-1 compares the site-specific EAB and LPZ χ/Q values with the ABWR EAB and LPZ χ/Q values from DCD Tier 2, Tables 15.6-3, "Instrument Line Break Accident Results"; 15.6-7, " Main Steamline Break Meteorology Parameters and Radiological Effects"; 15.6-13, "Loss of Coolant Accident Meteorology and Offsite Dose Results"; and 15.6-18, "Clean Up Water Line Break Meteorology and Dose Results". Table 15.6.5S-1, "Site-Specific χ /Q," shows that the ABWR DCD offsite χ/Q values are not exceeded.

2.3S.4.4.1.2 Control Room Dispersion Estimates

a. Atmospheric Dispersion Model

The applicant uses the computer code ARCON96 (NUREG/CR–6331, "Atmospheric Relative Concentrations in Building Wakes") to estimate χ/Q values at the CR and the TSC for potential accidental releases of radioactive material. The ARCON96 model implements the methodology outlined in RG 1.194.

The ARCON96 code estimates χ/Q values for various time-averaged periods ranging from 2 hours to 30 days. The meteorological input to ARCON96 consists of hourly values of wind speed, wind direction, and atmospheric stability class. The χ/Q values calculated through ARCON96 are based on the theoretical assumption that material released into the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the release points and receptors. The diffusion coefficients account for an enhanced dispersion under low wind speed conditions and in building wakes.

The hourly meteorological data are used to calculate hourly relative concentrations. The hourly relative concentrations are then combined to estimate concentrations ranging in duration from 2 hours to 30 days. Cumulative frequency distributions are prepared from the average relative concentrations and the relative concentrations that are exceeded no more than five percent of the time for each averaging period are selected.

b. Meteorological Data Input

The meteorological input to ARCON96 used by the applicant consisted of hourly onsite wind speed, wind direction, and atmospheric stability data from the years 1997, 1999, and 2000. The wind data were obtained from the 10-m and 60-m (33-ft and 197-ft) levels of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-T) measurements taken between the 60-m and 10-m (197-ft and 33-ft) levels on the onsite meteorological tower.

As discussed in SER Section 2.3S.3, the staff considers the 1997, 1999, and 2000 onsite meteorological database suitable for input to the ARCON96 model.

c. Diffusion Parameters

The diffusion coefficients used in ARCON96 have three components. The first component, the diffusion coefficient, is used in other NRC models such as PAVAN. The other two components are corrections to account for the enhanced dispersion under low wind speed conditions and in building wakes. These components are based on an analysis of diffusion data collected in various building wake diffusion experiments, under a wind range of meteorological conditions. Because the diffusion occurs at short distances within the plant's building complex, the ARCON96 diffusion parameters are not affected by nearby topographic features, such as bodies of water. Therefore, the staff found that the applicant's use of the ARCON96 diffusion parameter assumptions is acceptable.

d. Resulting Relative Concentrations

FSAR Tier 2, Figure 2.3S-23, is a map showing potential atmospheric accident release pathways and control room and TSC receptors. As discussed in ABWR DCD Tier 2, Subsection 15.6.5.5.3, the control room may be contaminated from two sources: the reactor building stack base and the turbine building truck doors. The applicant treats both the reactor building stack base and the turbine building truck doors as ground level sources. For STP, Units 3 and 4, each unit has two control room air intakes and one TSC air intake. The applicant treats these three intakes as receptors for the ARCON96 modeling. The applicant chose the highest χ/Q values among these three intakes for comparison to the ABWR control room χ/Q values from DCD Tier 2, Table 15.6-14, "Loss of Coolant Accident Meteorology and Control Room Dose Results," in compliance with COL License Information Item 2.11.

FSAR Tier 2, Table 2.3S-25, "ARCON96 χ /Q Values (sec/m3)," presents the resulting χ/Q values determined by the applicant's ARCON96 dispersion modeling at the control room and TSC air intakes for releases from the reactor building plant stack and turbine building truck doors. The staff issued RAI 02.03.04-1, asking the applicant to describe the inputs used to execute the ARCON96 atmospheric dispersion model, for each source-receptor combination, to derive the control room and TSC χ/Q values presented in Revision 0 to FSAR Tier 2, Table 2.3S-25. In its response to RAI 02.03.04-1, dated May 29, 2008 (ML081560702), the applicant described the inputs used to execute ARCON96 for the χ/Q values presented in Revision 0 to FSAR Tier 2, Table 2.3S-25.

The applicant revised the control room and TSC χ/Q values presented in FSAR Tier 2, Table 2.3S-25 in its response to RAI 15.00.03-1, dated October 26, 2009 (ML093030297), and then submitted a revised response to RAI 15.00.03-1, dated November 30, 2009 (ML093360204). In order to review the applicant's revised control room and TSC χ/Q values, the staff issued RAI 02.03.04-10, requesting that the applicant to provide the revised set of inputs used to rerun the ARCON96 model.

In its response to RAI 02.03.04-10, dated December 21, 2009 (ML093580191), provides a description of the revised inputs used to execute the ARCON96 dispersion model. The applicant stated that the revised inputs result from updated information regarding the location and specifications for the release points and receptors. The most signification revisions result from the following:

- Reduction of the release height of the plant stack from 76 m (249 ft) to 26.2 m (86 ft). (The release is assumed to occur at the stack base instead of the top of the stack in accordance with the release descriptions provided in the DCD Tier 2, Subsection 15.6.5.5.3.2.)

- Reduction of distances from sources to the TSC air intakes. (The TSC air intakes are conservatively assumed to be located at the TSC southwest corner for the reactor building stack releases and the TSC northwest corner for the turbine building truck door releases.)

The applicant also agrees in the response to RAI 02.03.04-10, to revise the turbine building truck door χ/Q values presented in FSAR Tier 2, Table 2.3S-25 to show three significant digits. This precision is necessary to compare site-specific χ/Q values to the DCD χ/Q values since the

DCD χ/Q values are presented to the third significant digit. The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed changes, per the applicant's commitment in the RAI response. Therefore, RAI 02.03.04-10 is resolved and closed.

The staff reviewed the applicant's inputs to the ARCON96 code and found them consistent with site configuration drawings and the guidance in RG 1.194. The staff confirmed the applicant's atmospheric dispersion estimates by running the ARCON96 computer model and generating the same results. The staff therefore accepts the control room and TSC χ/Q values presented by the applicant. Therefore, RAI 02.03.04-1 is resolved and closed.

In accordance with COL License Information Item 2.11, FSAR Tier 2, Table 15.6.5S-1, "Site-Specific χ /Q," compares the site-specific control room and TSC χ/Q values with the ABWR control room χ/Q values from DCD Tier 2, Table 15.6-14. Table 15.6.5S-1 concludes that with two exceptions, the ABWR DCD control room χ/Q values are not exceeded. The two exceptions are in regards to: (1) the calculated 4–30 day χ/Q value for a turbine building release ($9.15\times10^{-5}$ s/m$^3$) that exceeded the corresponding ABWR DCD χ/Q value ($8.53\times10^{-5}$ s/m$^3$); and (2) the calculated 4–30 day χ/Q value for a reactor building release ($5.59\times10^{-4}$ s/m$^3$) that exceeded the corresponding ABWR DCD χ/Q value ($5.12\times10^{-4}$ s/m$^3$). SER Section 15.6 discusses the consequences of these two exceptions.

#### 2.3S.4.4.2 Hazardous Material Releases

The atmospheric dispersion models used by the applicant to calculate atmospheric dispersion for hazardous material releases are discussed in FSAR Tier 2, Section 2.2S.3. SER Section 2.2.3 discusses the staff's technical evaluation of the applicant's dispersion estimates associated with accidental onsite and offsite hazardous material releases.

### *2.3S.4.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.3S.4.6 Conclusion*

The staff reviewed the application and found that the applicant has presented and substantiated information regarding short-term atmospheric dispersion estimates for accident releases. The staff reviewed the information and, for the reasons stated above, finds that the applicant's atmospheric dispersion estimates are acceptable and meet the relevant requirements of 10 CFR 100.21(c)(2). This finding is based on the conservative assessments of post-accident atmospheric dispersion conditions that have been made by the applicant and the staff from the applicant's meteorological data and appropriate dispersion models. These atmospheric dispersion estimates are appropriate for the assessment of consequences from radioactive releases for DBAs, in accordance with 10 CFR 52.79(a)(1)(vi) and GDC 19.

In addition, the staff compared the additional information in the application to the relevant NRC regulations and the guidance in Section 2.3.4 of NUREG–0800. The staff's review finds that the applicant has adequately addressed COL License Information Items 2.1 and 2.11 in accordance with Section 2.3.4 of NUREG–0800, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

## 2.3S.5 Long-Term Atmospheric Dispersion Estimates For Routine Releases

### *2.3S.5.1 Introduction*

This FSAR section addresses the atmospheric dispersion ($\chi$/Q or relative concentration) and dry deposition (D/Q or relative deposition) estimates to a distance of 80.5 km (50 mi) from the plant for routine releases of radiological effluents into the atmosphere during normal plant operation for use in annual average release limit calculations and offsite dose estimates.

### *2.3S.5.2 Summary of Application*

This site-specific supplement in the FSAR describes the following:

- Atmospheric dispersion models used to calculate concentrations in air and the amount of material deposited as a result of routine releases of radioactive material into the atmosphere.
- The characteristics assumed for each release point and the location of potential receptors for dose computations.
- Meteorological data and other assumptions used as inputs to the atmospheric dispersion models.
- Diffusion parameters ($\sigma_z$).
- Relative concentration factors and relative deposition factors used to assess the consequences of routine airborne radioactive releases.

In addition, in FSAR Section 2.3S.5, the applicant provides the following:

*COL License Information Item*

- COL License Information Item 2.12 Long-Term Atmospheric Dispersion Estimates For Routine Releases

This site-specific supplement addresses COL License Information Item 2.12, from the certified ABWR DCD, which states that COL applicants will provide annual average atmospheric dispersion values for routine releases for the NRC to review.

### *2.3S.5.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the long-term atmospheric dispersion estimates for routine releases, and the associated acceptance criteria are in Section 2.3.5 of NUREG–0800. In particular, the regulatory requirements are 10 CFR Part 20, 10 CFR 50.34a, 10 CFR Part 50 Appendix I and 10 CFR 100.21.

The staff considered the following regulatory requirements in reviewing the applicant's discussion of long-term atmospheric dispersion estimates:

- 10 CFR Part 20, Subpart D, with respect to establishing atmospheric dispersion site characteristics for demonstrating compliance with dose limits for individual members of the public.

- 10 CFR 50.34a and Sections II.B, II.C and II.D of Appendix I of 10 CFR Part 50, with respect to establishing atmospheric dispersion site characteristics for evaluating the numerical guides for design objectives and limiting conditions for operation to meet the requirements that radioactive material in effluents released to unrestricted areas be kept as low as is reasonably achievable.

- 10 CFR 100.21(c)(1), with respect to establishing atmospheric dispersion site characteristics so that radiological effluent release limits associated with normal operation can be met for any individual located offsite.

NUREG–0800, Section 2.3.5 specifies that an application meets the above requirements if the application provides the following information:

- A detailed description of the atmospheric dispersion and deposition models used by the applicant to calculate annual average concentrations in the air and the amount of material deposited as a result of routine releases of radioactive materials into the atmosphere.

- A discussion of atmospheric diffusion parameters, such as a vertical plume spread ($\sigma_z$), as a function of distance, topography, and atmospheric conditions.

- Meteorological data summaries (onsite and regional) used as input to the dispersion and deposition models.

- Points of routine release of radioactive material into the atmosphere, including the characteristics (e.g., location and release mode) of each release point.

- The specific location of potential receptors of interest (e.g., nearest vegetable garden, nearest resident, nearest milk animal, and nearest meat cow in each 22½-degree direction sector within a 8-km [5-mi] radius of the site).

- The $\chi$/Q and D/Q values to be used for assessing the consequences of routine airborne radiological releases described in Regulatory Position C.I.2.3.5.2 of RG 1.206:

  (1) Maximum annual average $\chi$/Q values and D/Q values at or beyond the site boundary and at specific locations of potential receptors of interest utilizing appropriate meteorological data for each routine venting location, and

  (2) Estimates of annual average $\chi$/Q values and D/Q values for 16 radial sectors to a distance of 80.5 km (50 mi) from the plant using appropriate meteorological data.

In addition, the long-term atmospheric dispersion estimates for routine releases should be consistent with appropriate sections from the following RGs:

- RG 1.23 provides criteria for an acceptable onsite meteorological measurements program; the program data are used as inputs to atmospheric dispersion models.

- RG 1.109, Revision 2, "Calculation of Annual Doses to Man from Routine Releases of Reactor Effluents for the Purpose of Evaluating Compliance with 10 CFR Part 50, Appendix I," presents criteria for identifying specific receptors of interest.

- RG 1.111, Revision 1, "Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors," provides acceptable methods for characterizing atmospheric transport and diffusion conditions and for evaluating the consequences of routine effluent releases.

- RG 1.112, Revision 1, "Calculation of Releases of Radioactive Materials in Gaseous and Liquid Effluents from Light-Water-Cooled Power Reactors," provides criteria for identifying release points and release characteristics.

When independently assessing the veracity of the information presented by the applicant in FSAR Tier 2, Section 2.3S.5, the staff applied the same methodologies, models, and techniques cited above.

### *2.3S.5.4 Technical Evaluation*

The staff reviewed the application and the applicant's responses to RAIs to verify the accuracy, completeness, and sufficiency of the information presented by the applicant regarding long-term atmospheric dispersion estimates for routine releases. The staff followed the procedures described in Section 2.3.5 of NUREG–0800 as part of this review.

The staff reviewed the following information in the COL FSAR:

*COL License Information Item*

- COL License Information Item 2.12 Long-Term Atmospheric Dispersion Estimates For Routine Releases

The staff's review of the "Long-Term Atmospheric Dispersion Estimates for Routine Releases," is summarized below.

#### 2.3S.5.4.1 Atmospheric Dispersion Model

The applicant uses the NRC-sponsored computer code XOQDOQ (described in NUREG/CR-2919, "XOQDOQ Computer Program for the Meteorological Evaluation of Routine Effluent Releases at Nuclear Power Stations") to estimate $\chi$/Q and D/Q values resulting from routine releases. The XOQDOQ model implements the constant mean wind direction model methodology outlined in RG 1.111.

The XOQDOQ model is a straight-line Gaussian plume model based on the theoretical assumption that material released into the atmosphere will be normally distributed (Gaussian) about the plume centerline. In predictions of $\chi$/Q and D/Q values for long time periods (i.e., annual averages), the plume's horizontal distribution is assumed to be evenly distributed within the downwind direction sector (e.g., "sector averaging"). A straight-line trajectory is assumed between the release point and all receptors.

Because geographic features such as hills, valleys, and large bodies of water can potentially influence dispersion and airflow patterns, terrain recirculation factors can be used to adjust the results of a straight-line trajectory model such as XOQDOQ to account for terrain-induced flows, recirculation, or stagnation. The staff issued RAI 02.03.05-1, asking the applicant to discuss the influence of the Gulf of Mexico and the resulting land and sea breezes on the routine release atmospheric dispersion estimates in FSAR Tier 2, Section 2.3S.5. In its response to RAI 02.03.05-1, dated May 29, 2008 (ML081560702), the applicant stated that sea breezes from the Gulf of Mexico will tend to increase routine release χ/Q and D/Q values due to the potential for local air recirculation. In order to account for possible sea breeze and land breeze effects from Matagorda Bay and the Gulf of Mexico on the long-term atmospheric dispersion estimates for routine releases, the applicant used default open terrain correction factors from the XOQDOQ dispersion model. This calculation means that all χ/Q and D/Q values out to a distance of 1 km (0.6 mi) are multiplied by a factor of four and all χ/Q and D/Q values between 1 and 10 km (0.6 and 6.2 mi) are multiplied by a factor that deceases logarithmically from four at 1 km (0.6 mi) to one at 10 km (6.2 mi).

The staff agreed with the applicant that the use of the default XOQDOQ open terrain correction factors conservatively account for possible recirculation due to land-water boundaries at the proposed STP, Units 3 and 4, site. Therefore, RAI 02.03.05-1 is resolved and closed.

### 2.3S.5.4.2 Release Characteristics and Receptors

The applicant models one ground level release point that assumes a minimum building cross-sectional area of 2,134 $m^2$ (22,970 $ft^2$) and a building height of 37.7 m (123.7 ft). The applicant stated that the minimum building cross-sectional area and height are based on the dimensions of the ABWR reactor building structure.

ABWR DCD Tier 2, Section 11.3.10, states that the primary release point for the ABWR plant is the reactor building stack, which is a roof-mounted, 2.4-m (7.9 ft) diameter circular stack extending to a height of 76 m (149.3 ft) above ground level. A ground level release is a conservative assumption that a flat-terrain site such as STP, Units 3 and 4, results in higher χ/Q and D/Q values when compared to a mixed-mode (e.g., part-time ground, part-time elevated) release or a 100-percent elevated release, as discussed in RG 1.111. A ground level release assumption is therefore acceptable to the staff.

Revision 0 to the FSAR states that the applicant executed XOQDOQ using the shortest distance from either the STP, Unit 3, reactor building to the EAB or the STP, Unit 4, reactor building to the EAB for each downwind sector. Likewise, the applicant also states in Revision 0 to the FSAR that the shortest distances are used from the STP, Units 3 and 4, reactor buildings to the various receptors of interest (i.e., nearest resident, meat animal, and vegetable garden) in each directional sector. The staff asked the applicant in RAI 02.03.05-8, to review an apparent discrepancy between the special receptor distances listed in FSAR Tier 2, Table 2.3S-26, "Distances from the Release Points to Sensitive Receptors," and the Land Use Census results reported in the STP 2006, Annual Environmental Operating Report. In its response to RAI 02.03.05-8, dated December 18, 2008 (ML083570395), states that the long-term atmospheric dispersion estimates for routine releases are being recalculated, and the special receptor distances listed in FSAR Tier 2, Table 2.3S-26 will be revised to be consistent with information in Revision 15 of the STP Offsite Dose Calculation Manual (ODCM). The ODCM reflects the distances to the receptors of interest reported in the Land Use Census results presented in the STP, Units 1 and 2, 2006, Radiological Environmental Operating Report. The revised special receptor distances listed in FSAR Tier 2, Table 2.3S-26, along with the resulting revised χ/Q and D/Q values, are provided by the applicant in its response to RAI 02.03.04-5, dated January 28, 2009 (ML090300648). Therefore, RAI 02.03.05-8 is resolved and closed.

In its response to RAI 02.03.04-5, dated January 28, 2009 (ML090300648), the applicant revised the approach for calculating distances to the EAB and receptors of interest by defining a power block envelope that encloses the STP, Units 3 and 4, reactor buildings and turbine buildings. The applicant then determines the shortest distances from the power block envelope to the EAB and various receptors of interest for each directional sector. The applicant uses these revised distances in the XOQDOQ analysis to propose updates to the EAB and special receptor χ/Q values. The applicant proposed revisions to FSAR Tier 2, Section 2.3S.5 to reflect this revised approach for calculating distances to the EAB and special receptors and to present the revised set of routine release χ/Q values.

In its revised response to RAI 02.03.04-5, dated July 30, 2009 (ML092150966), the applicant stated that the long-term atmospheric dispersion estimates are being revised based on a release from either the STP, Unit 3 or 4, reactor building stack, whichever is closest to the site boundary and receptors of interest, instead of from the power block envelope. The applicant updated the receptor distances to be consistent with Revision 15 of the ODCM and recalculated the long-term χ/Q and D/Q values. The revised long-term χ/Q and D/Q values are then incorporated into Revision 3 to the FSAR. Therefore, RAI 02.03.04-5 is resolved and closed. However, the applicant's revised response to RAI 02.03.04-5, presented, for the first time, maximum annual χ/Q and D/Q values for the site boundary as well as for the EAB.
In RAI 02.03.05-11, the staff requested the applicant to revise the FSAR to provide the downwind distances to the site boundary and EAB in each sector used to derive the revised set of maximum annual site boundary and EAB χ/Q and D/Q values. In its response to RAI 02.03.05-11, dated October 29, 2009 (ML093430299), the applicant provided a proposed revision to FSAR Tier 2, Section 2.3S.5 which includes tables that provide the requested downwind distances to the EAB and site boundary in each sector. The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed changes to Section 2.3S.5. Therefore, RAI 02.03.05-11 is resolved and closed.

The staff noticed that Revision 3 of FSAR Tier 2, Section 2.3S.5.2 states that the maximum annual average no-decay χ/Q value for the EAB is $8.1 \times 10^{-6}$ s/m$^3$ in the northwest sector at a distance of 1.11 km (0.69 mi). This appears to conflict with the information presented in FSAR Tier 2, Table 2.3S-27, which shows that the maximum no-decay χ/Q value for the EAB is $1.5 \times 10^{-5}$ s/m$^3$ in the northwest sector at a distance of 0.84 km (0.52 mi). The staff issued RAI 02.03.05-12, requesting that the applicant revise the FSAR to address this apparent conflict. The applicant proposed a revision to the FSAR resolving this conflict in its response to RAI 02.03.05-12, dated March 1, 2010 (ML100620824). The staff confirmed that FSAR Tier 2, Revision 4 includes the proposed changes, per the applicant's commitment in the RAI response. Therefore, RAI 02.03.05-12 is resolved and closed.

Note that no residential milk cows were identified within 8 km (5 mi) of the STP site.
The applicant assumed that all residents have a vegetable garden and are fattening a calf for residential consumption. The staff found these to be conservative assumptions and therefore acceptable.

#### 2.3S.5.4.3 Meteorological Data Input

The meteorological input to the XOQDOQ model consists of a joint frequency distribution of wind speed, wind direction, and atmospheric stability based on hourly onsite data from the three-year period of 1997, 1999, and 2000. The wind data were obtained from the 10-m (33-ft) level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-T) measurements taken between the 60-m and 10-m (197-ft and 33-ft) levels on the onsite meteorological tower.

As discussed in SER Section 2.3S.3, the staff considers the 1997, 1999, and 2000 onsite meteorological database suitable for input to the XOQDOQ model.

#### 2.3S.5.4.4 Diffusion Parameters

The applicant chooses to implement the diffusion parameter assumptions, outlined in RG 1.111, as a function of atmospheric stability for the XOQDOQ model runs. Due to the location and size of the main cooling reservoir, overwater trajectories in the south-southeast to the south-southwest downwind sectors average approximately 5 km (3 mi). The staff asked the applicant in RAI 02.03.05-6 to describe the impact of reduced surface roughness resulting from the main cooling reservoir over-water trajectories on the resulting long-term, offsite atmospheric dispersion estimates. In its response to RAI 02.03.05-6, dated November 20, 2008 (ML08390340), the applicant stated that the reduced surface roughness induced by the main cooling reservoir will result in less mechanical turbulence and higher χ/Q values. The decrease in mechanical turbulence is offset by an increase in thermal turbulence due to the heating from below the overwater trajectories. The applicant also states that reduced surface roughness will also increase ambient wind speed slightly, thus increasing dispersion. The net effect leads to minimal changes in annual average χ/Q values. Therefore, the applicant’s use of diffusion parameter assumptions outlined in RG 1.111 is acceptable to the staff, and RAI 02.03.05-6 is resolved and closed.

2.3S.5.4.5 Resulting Relative Concentration and Relative Deposition Factors

FSAR Tier 2, Table 2.3S-27, “XOQDOQ-Predicted Maximum χ/Q and (D/Q) Values at Receptors of Interest,” lists the long-term atmospheric dispersion and deposition estimates for the EAB, site boundary, and special receptors of interest that the applicant derived from the XOQDOQ modeling results. The χ/Q values in FSAR Tier 2, Table 2.3S-27 reflect several plume radioactive decay and deposition scenarios. Regulatory Position C.3 of RG 1.111 states that radioactive decay and dry deposition should be considered in radiological impact evaluations of potential annual radiation doses to the public that result from routine releases of radioactive materials in gaseous effluents. Regulatory Position C.3.a of RG 1.111 states that an overall half-life of 2.26 days is acceptable for evaluating the radioactive decay of short-lived noble gases, and an overall half-life of 8 days is acceptable for evaluating the radioactive decay for all iodine released into the atmosphere. Definitions for the χ/Q categories listed in the headings of FSAR Tier 2, Table 2.3S-27 are as follows:

- No Decay χ/Q values are χ/Q values used to evaluate ground level concentrations of long-lived noble gases, tritium, and carbon-14. The plume is assumed to travel downwind, without undergoing dry deposition or radioactive decay.

- 2.26-Day Decay χ/Q values are χ/Q values used to evaluate ground-level concentrations of short-lived noble gases. The plume is assumed to travel downwind, without undergoing dry deposition, but is decayed, assuming a half-life of 2.26 days, based on the half-life of xenon-133m.

- 8.00-Day Decay χ/Q values are χ/Q values used to evaluate ground level concentrations of radioiodine and particulates. The plume is assumed to travel downwind, with dry deposition, and is decayed, assuming a half-life of 8.00 days based on the half-life of iodine-131.

In RAI 02.03.05-5, the staff asked the applicant to clarify whether the no-decay and 2.26-day decay χ/Q values in FSAR Tier 2, Table 2.3S-27 assume no dry deposition, and whether the 8-day decay χ/Q values in the same table assume dry deposition. In its response to RAI 02.03.05-5, dated June 12, 2008 (ML081710126), the applicant confirmed these assumptions. The applicant revised the FSAR in Revision 3 to state that the no-decay and 2.26-day decay χ/Q values in Table 2.3S-27 assume no dry deposition, and the 8-day decay χ/Q values in the same table assume dry deposition. Therefore, RAI 02.03.05-5 is resolved and closed.

FSAR Tier 2, Tables 2.3S-28, "XOQDOQ-Predicted Annual Averate χ /Q Values at the Standard Radial Distances and Distance-Segment Boundaries," and 2.3S-29, "XOQDOQ-Predicted Annual Average D/Q Values at the Standard Radial Distances and Distance-Segment Boundaries," list the applicant's long-term atmospheric dispersion and deposition estimates for all 16 radial sectors from the site boundary to a distance of 80.5 km (50 mi) from the proposed facility.

The staff reviewed the XOQDOQ computer code inputs and outputs provided by the applicant in the response to environmental RAI 05.04.02-1, dated September 22, 2009 (ML092710535), and reran the model using the applicant's revised distances to the EAB, site boundary, and receptors of interest provided in the responses to RAIs 02.03.04-5 and 02.03.05-11. The staff's results were consistent with the applicant's results presented in the FSAR.

### *2.3S.5.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.3S.5.6 Conclusion*

The staff reviewed the application and found that the applicant has presented and substantiated information regarding long-term atmospheric dispersion estimates for routine releases. The staff reviewed the information and, for the reasons stated above, concluded that the applicant's atmospheric dispersion estimates are acceptable and meet the relevant requirements of 10 CFR 100.21(c)(1). Representative atmospheric dispersion and deposition factors have been calculated for 16 radial sectors from the site boundary to a distance of 80.5 km (50 mi) as well as for specific locations of potential receptors of interest. The characterization of atmospheric dispersion and deposition conditions is appropriate for the evaluation to demonstrate compliance with the numerical guides for doses in Subpart D of 10 CFR Part 20 and Appendix I to 10 CFR Part 50.

In addition, the staff compared the additional information in the application to the relevant NRC regulations and the guidance in Section 2.3.5 of NUREG–0800. The staff's review finds that the applicant has adequately addressed COL License Information Item 2.12, in accordance with Section 2.3.5 of NUREG–0800. The staff determined that applicant has addressed the relevant information, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

## 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, the 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 SER in accordance with the review procedures described in NUREG–0800 using information presented in Section 2.4S of the STP, Units 3 and 4, COL FSAR, which references Revision 4 to the ABWR DCD; the applicant responses to 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 Revision 12, 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 RGs for the identified acceptance criteria:

- 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.

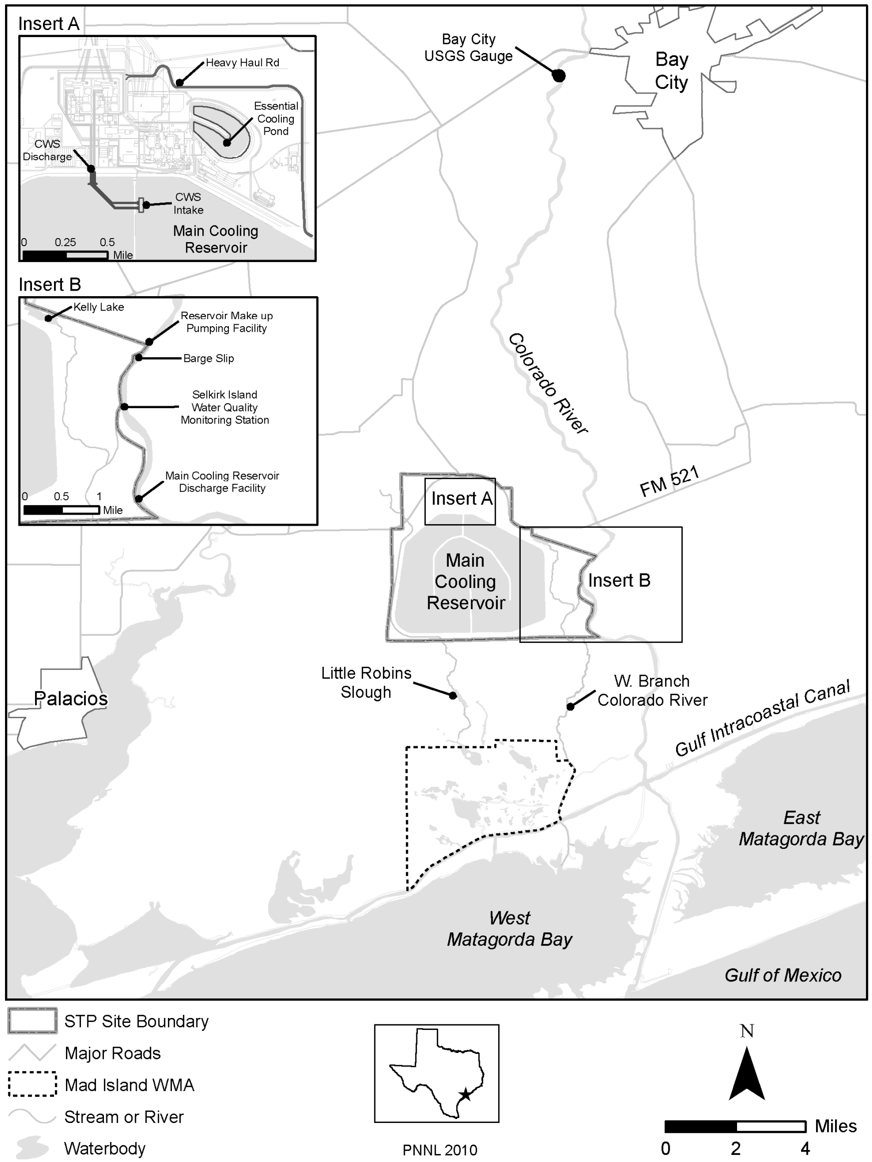


**Figure 2.4S.1-1 Map Showing the Location of the STP Site**

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 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 (0.3 m) 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.

*The 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 STP, 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.

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, "Site Map of the General Area of STP 3 & 4"). 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.

*<u>Information Submitted by Applicant</u>*

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.

<u>The Colorado River Basin</u>

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 one 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 stated that the expected frequency of occurrence of major hurricanes is approximately once every three 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.

*The 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 (SER 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.

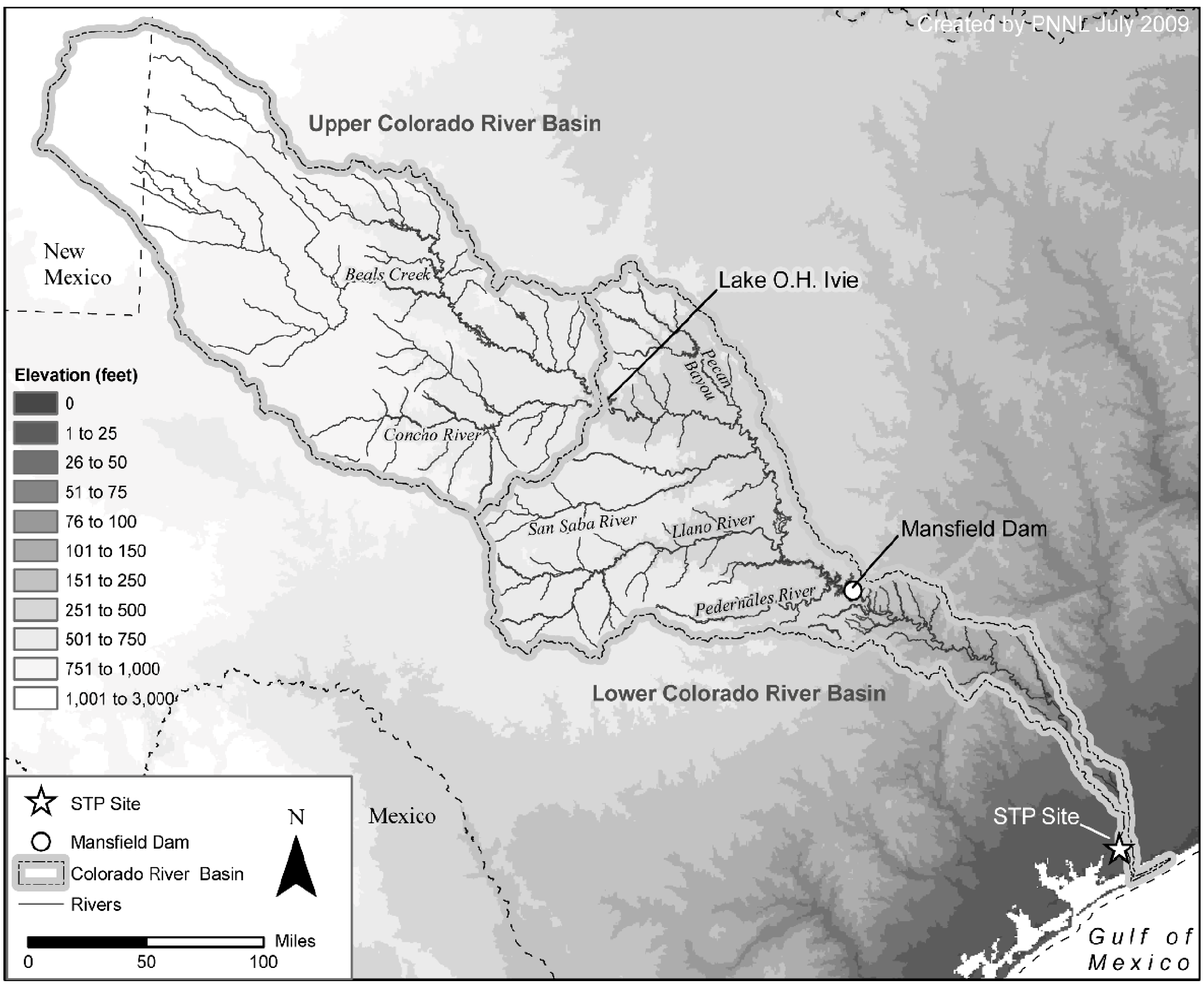


**Figure 2.4S.1-2 The Colorado River Basin (The point of demarcation between the upper and the lower basin is Lake O.H. Ivie; Mansfield Dam and the STP site are also shown)**

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 STP, 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 (SER 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 STP, 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 STP, 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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 stated 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) 521 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, “Floods,” 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.

[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.

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.

*The 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 SER 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 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.

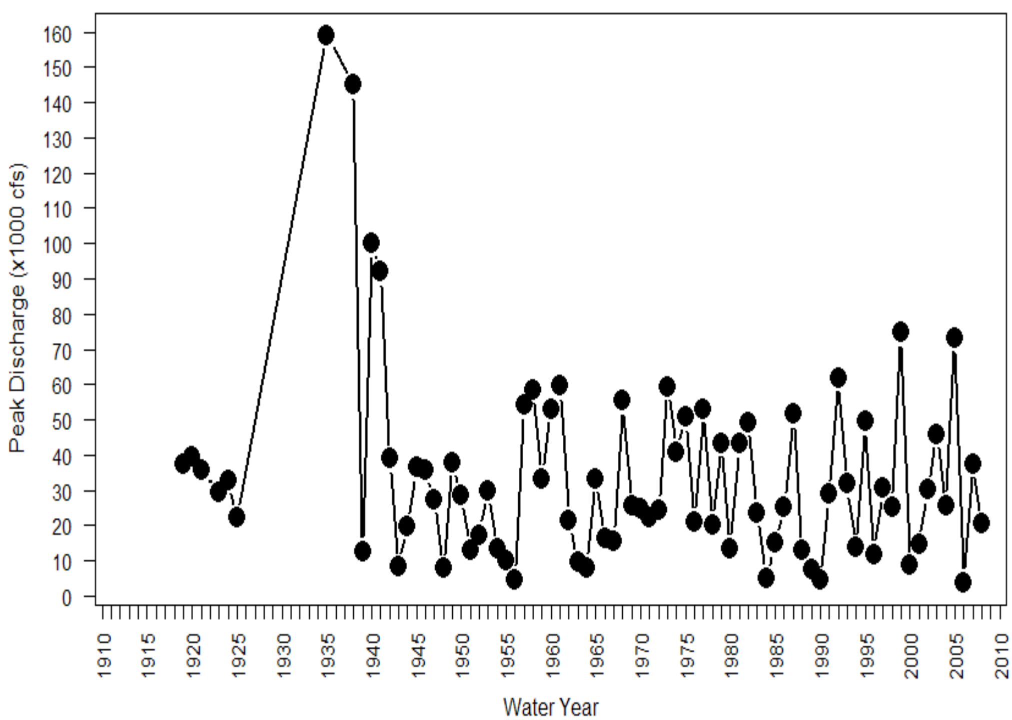


**Figure 2.4S.2-1 Peak Stream-Flow Discharge in the Colorado River at the Wharton USGS Gauge**

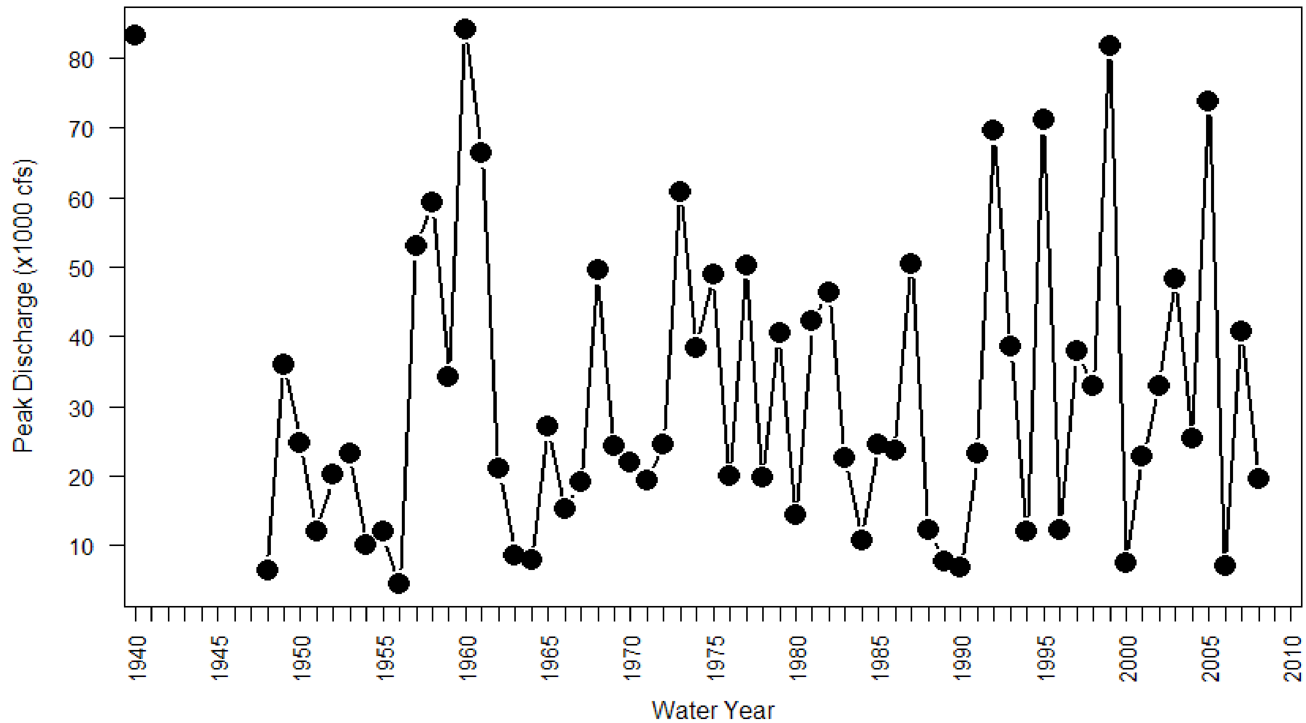


**Figure 2.4S.2-2. Peak Stream-Flow Discharge in the Colorado River at the Bay City USGS Gauge**

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.

*The 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. 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-mi2 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 |

Local Drainage Components and Subbasins

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, "PMP Peak Discharges in STP 3 & 4 Subbasins and Drainage Elements," 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, "Extents and Locations of Channel Crosssections"). 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.

*The 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 its response to RAI 02.04.02-1,dated June 12, 2008 (ML081710126), the applicant stated 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, "Flood Protection for Nuclear Power Plants."

The staff reviewed the applicant's response and determined that the safety-related SSCs for STP, 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 resolved and 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, "Probable Maximum Flood (PMF) on the Streams and Rivers," and 2.4.4, "Potential Dam Failures."

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 4surface 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 its response to RAI 02.04.02-3, dated August 12, 2008 (ML091811141), the applicant refered 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 stated in its 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 stated 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.

In its respondse to RAI 02.04.02-3, dated August 12, 2008 (ML091811141), the applicant responsed to subparts (b), (c), and (e) are relevant to the discussions in this subsection.

The applicant stated 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 noted 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 stated 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 STP, 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 stated 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 resolved and closed. In its 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 RAI 02.04.02-1 is resolved and 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 its response to RAI 02.04.02-4, dated July 9, 2008 (ML081960070), the applicant stated 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 noted 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 stated 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 resolved and 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, 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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 noted 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.

*The 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).

*The 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 stated in FSAR, Subsection 2.4S.3.4.2.1. In its response to RAI 02.04.03-7(c), dated July 2, 2008 (ML081890239), the applicant stated 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 resolved and 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.

*The 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 its response to RAI 02.04.03-7(b) (ML081890239), dated July 2, 2008, the applicant stated 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 resolved and 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, "PMF and SPF values at Mansfield Dam," 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 concluded 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).

*The 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 stated 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 stated 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.

*The 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. In its response to RAI 02.04.03-6, dated July 9, 2008 (ML081960070), the applicant stated 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 concluded 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 resolved and 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 concluded that PMF water surface elevations combined with wind waves will not be the controlling scenario at the STP, Units 3 and 4, site.

*The 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, and
- 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 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 finds 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 finds 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.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, of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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 and
- 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 stated 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 its response to RAI 02.04.08-01, dated August 27, 2008 (ML082490086), the applicant described 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 concluded 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>*The 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, Units 1 and 2, SER. In the STP, Units 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, Units 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 STP, 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, "Summary of the 68 Dams in Colorado River Basin with 5,000 AF or More Storage Capacity," 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 | Baseline Halff | Values increased by | To account for increased |

| Model Element | Halff (2002) | FSAR | Rationale |
|---|---|---|---|
| within 4 mi downstream of Buchanan and Mansfield dams[(a)] | (2002) | a factor of 2 compared to those used by Halff Associates, Inc. (2002) for Manning's n | 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 stated 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 (127.1 m) respectively, and the peak discharge to be 1,773 $m^3/s$ (62,600 cfs). The applicant stated that the peak discharge predicted by the FLDWAV model using the breach width of 417 ft (127.1 m) 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.

*The 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, and
- 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 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 RAI 02.04.04-13 and 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. Therefore, RAI 02.04.04-13 and Open Item 2.4.4-1, are resolved 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 STP, 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 1.88x10$^8$ 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 x 10$^8$ to 3.70 x 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 |

| Parameter | Teton Dam | Martin Cooling Pond |
|---|---|---|
| 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 stated 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 stated 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, “Locations for RMA2 Modeling Results,” 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) |

| 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** |
| 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<br>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, "STP Site Layout," 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 its revised response to RAI 02.04.04-15, dated November 22, 2010 (ML111150106), the applicant described 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 stated 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.

Hydrodynamic Forces

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. In its response to RAI 02.04.04-15, dated March 28, 2011 (ML110890901), the applicant provided 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 concluded 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 concluded 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.

*The 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. In its response to RAI 02.04.04-9, dated January 28, 2009, and February 23 of 2009 (ML090300648 and ML090710301),[1] the applicant stated 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 stated 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.

[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.

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 resolved and 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, 2009, and February 23, 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 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 its response to RAI 02.04.04-11, dated August 26, 2009 (ML092430134), the applicant stated 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 noted 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 its response to RAI 02.04.04-12, dated August 26, 2009 (ML092430134), the applicant stated 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 noted 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 resolved and closed. In its 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. The staff confirmed that FSAR Revision 6 incorporated this information into the FSAR. Therefore, RAI 02.04.04-1 is resolved and 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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:

<u>*COL License Information Items*</u>

- 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 RGs 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), and

- 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 "Probable Maximum Winds and Associated Meteorological Parameters" and Table 2.4S.5-2, "Probable Maximum Hurricane Characteristics" (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. In its 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), the applicant refered 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 concluded 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.

*The 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, "Probable Maximum Hurricane Scenarios and Probable Maximum Surge Elevations" (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 resolved and 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) | $7.1 \times 10^{-5}$ | |
| 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 | $P_w - \Delta P$ |
| $\Delta P$ (cm / in. Hg) | 9.98 / 3.93 | Figure 2.3 |

| Parameter (units) | Value | Source in NOAA (1979) |
|---|---|---|
| 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 stated 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 its 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 resolved and closed.

2.4S.5.4.2 Surge and Seiche Water Levels

<u>*Information Submitted by Applicant*</u>

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 stated 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. In its response to RAI 02.04.05-11, dated November 22, 2010 (ML103330369), the applicant stated that it had performed ADCIRC simulations in addition to the scenario presented to the staff during the site audit on August 31, 2010, and September 1, 2010.

The applicant's response to RAI 02.04.05-11, part (1) describes the ADCIRC model. The applicant stated 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 stated 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 stated 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 stated 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 noted 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 stated 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 stated 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 stated 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 noted that the ADCIRC is the standard coastal model used by the USACE.

In its response to RAI 02.04.05-11, part (2), the applicant provides a detailed description of data sets used with the ADCIRC. The applicant stated 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 stated 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 stated 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 stated 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 stated 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 stated 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 noted 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 stated 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 stated 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 stated 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 noted 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 concluded 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 its response to RAI 02.04.05-10 (ML102100047), dated July 27, 2010, the applicant stated 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 $\Delta$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 its response to RAI 02.04.05-11, part (5), the applicant reported 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 noted 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 concluded 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 noted 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 its response to RAI 02.04.05-11, part (6), the applicant stated 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.

In its response to RAI 02.04.05-11, part (7) the applicant stated 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 stated 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.

*The 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. In its response to RAI 02.04.05-1, dated June 26, 2008 (ML081970231), the applicant 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 resolved and 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." In its response to RAI 02.04.05-2, dated August 27, 2008 (ML082490086), the applicant stated 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 resolved and 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. In its response to RAI 02.04.05-3, dated August 27, 2008, the applicant stated 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. Therefore, RAI 02.04.05-3 is resolved and 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. In its response to RAI 02.04.05-5, dated September 4, 2008 (ML082530449), the applicant stated 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 resolved and 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. In its response to RAI 02.04.05-6, dated September 4, 2008, the applicant refered 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 concluded 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 resolved and 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–$\Delta$P 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. In its response to RAI 02.04.05-9, dated September 16, 2009 (ML092610376), the applicant 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.

**The 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); and
- 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.

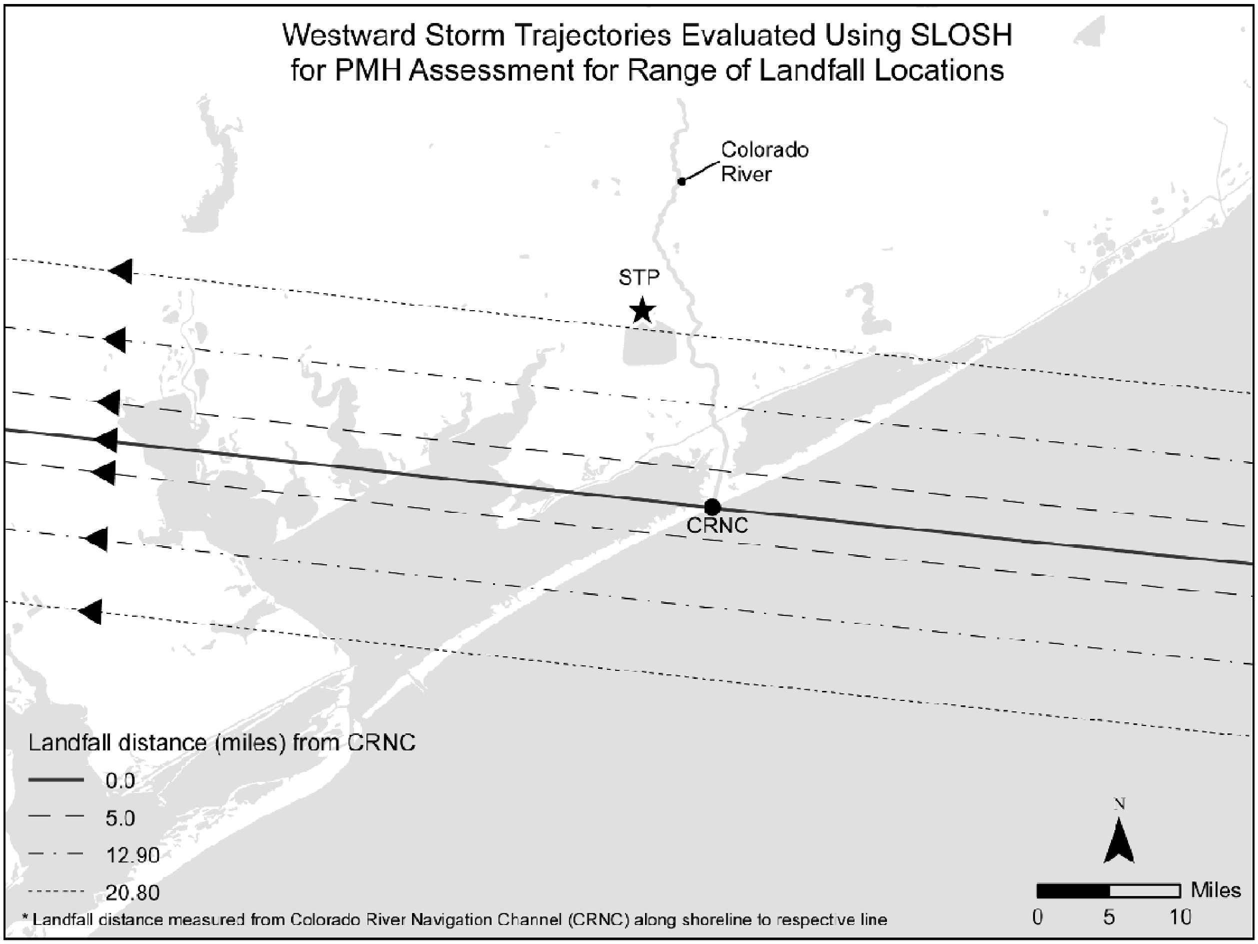


**Figure 2.4S.5-1 Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations**

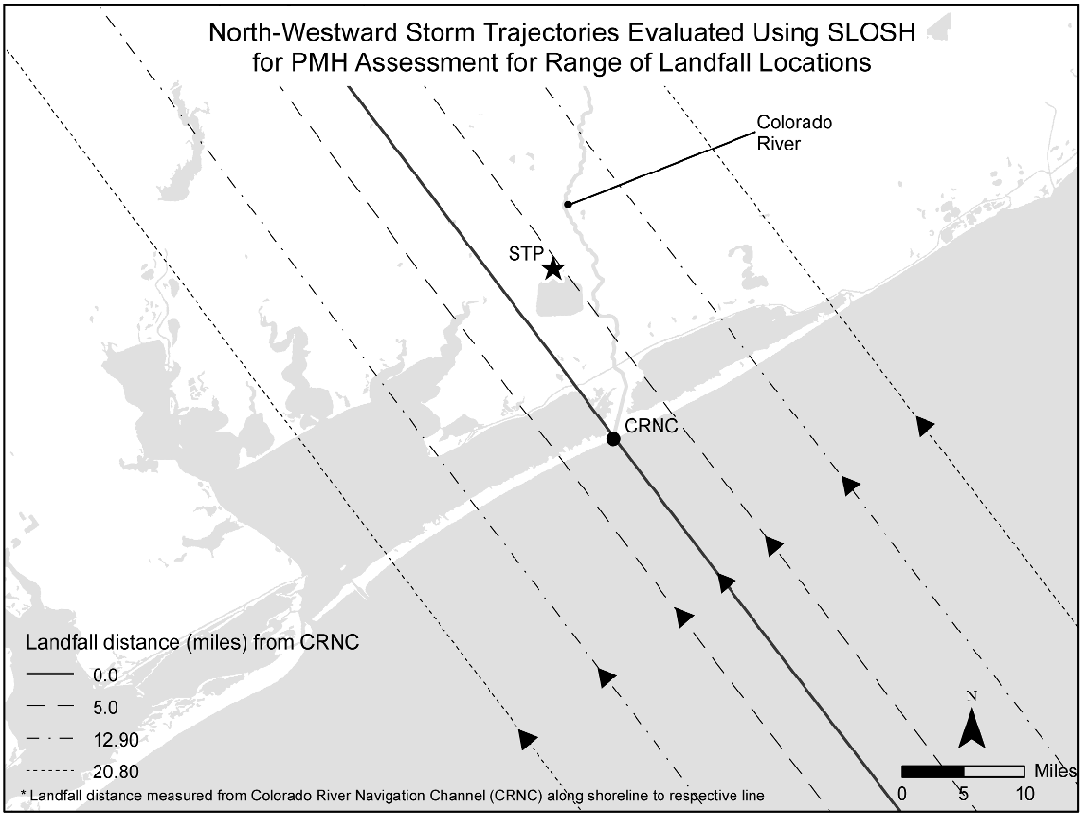


**Figure 2.4S.5-2 North-Westward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations**

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.

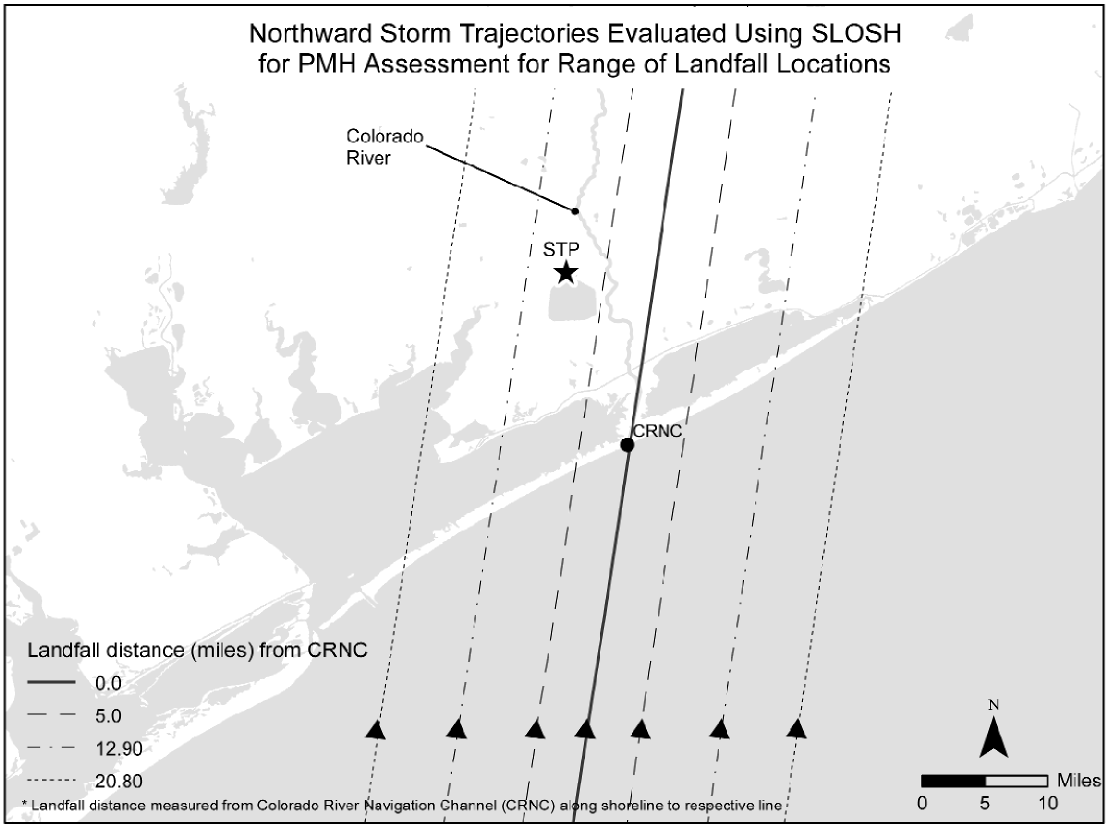


**Figure 2.4S.5-3 Northward PMH Storm Tracks Used in the NRC Staff's SLOSH Simulations**

**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 its response to RAI 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 °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 resolved and closed.

**The 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.

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 $10^{-13}$).

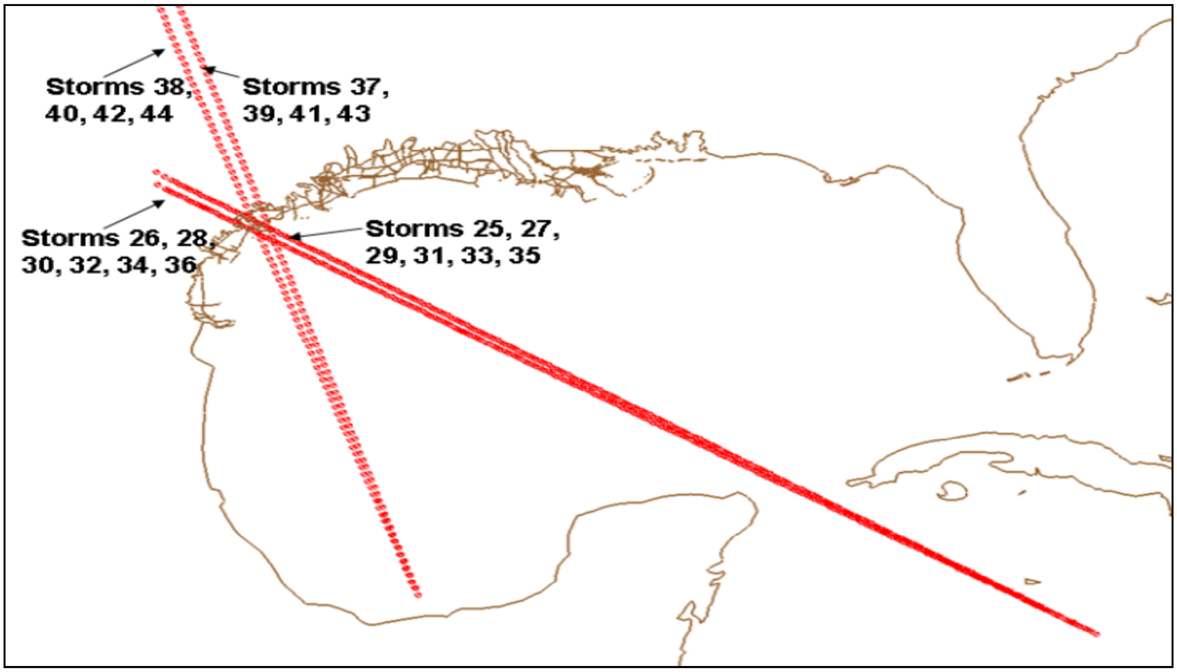


**Figure 2.4S.5-4 Storm Tracks Developed with the Maximum Wind Speeds over the Point of Interest**

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 stated that the maximum PMSS water surface elevation of 8.9 m (29.3 ft) MSL includes wind-wave effects.

*The 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.

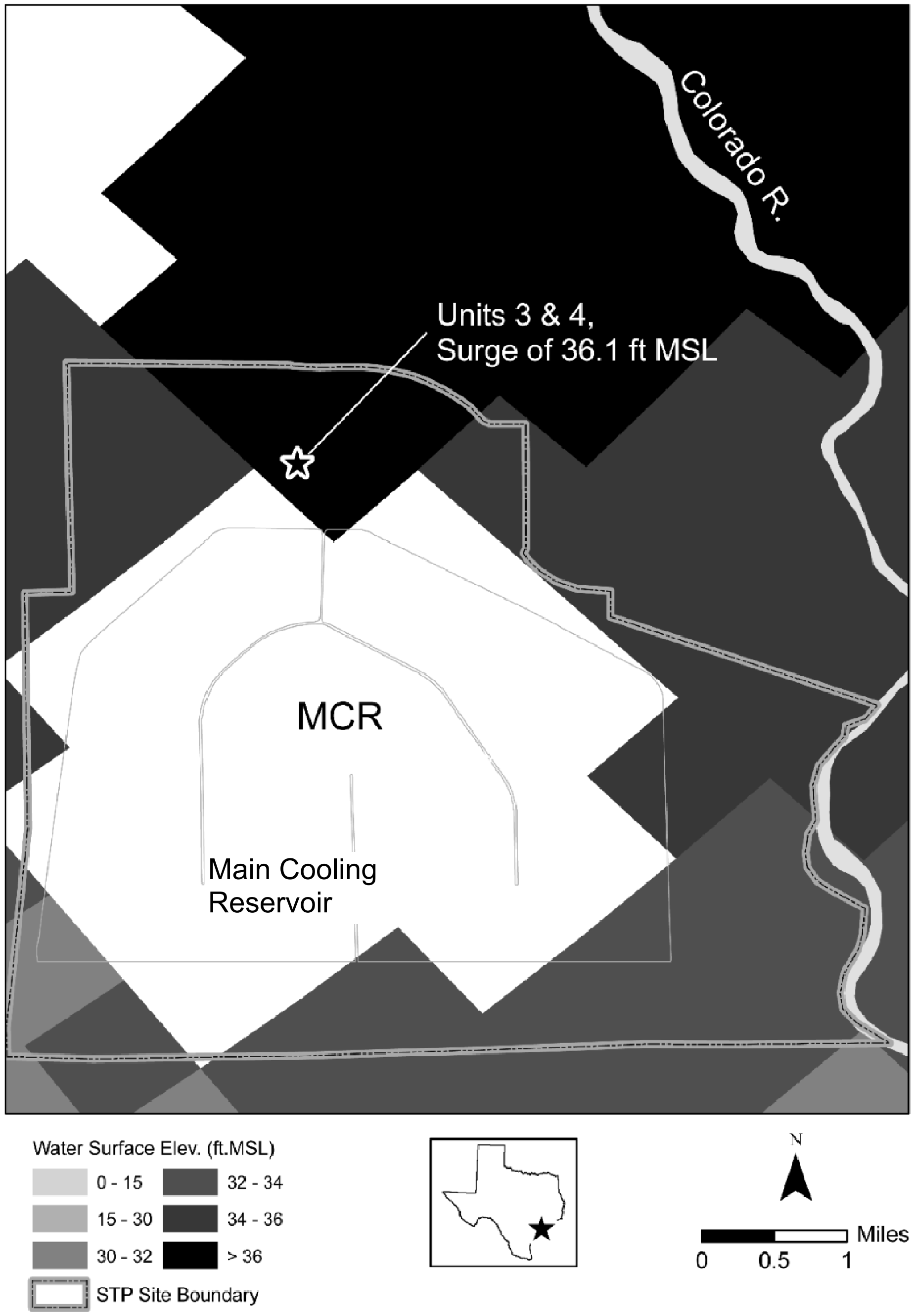


**Figure 2.4S.5-5 NRC Staff-Estimated PMSS Water Surface Elevations at the STP Site**

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.

*The Staff's Technical Evaluation*

The applicant stated 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 its response to RAI 02.04.05-7, dated August 12, 2008 (ML082270381), the applicant stated 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, "Potential Dam Failures."

*The 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 its response to RAI 02.04.050-11, dated November 22, 2010 (ML111510810), the applicant provided 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, "Design of Reinforced Grass Waterways (CIRIA Project Report)" by Hewlett, H.W.M., Boorman, L.A., and Bramley, M.E., 1987, states 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). 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, the staff finds Open Item 2.4.5-1, to be resolved and 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. The 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 concluded 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 RGs 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*

The 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 stated 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 its response to RAI 02.04.06-1, dated December 4, 2008 ML083460084) and in the FSAR.

In RAI 02.04.06-1, the staff 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.

*The 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: In its response to RAI 02.04.06-1, dated December 4, 2008 (ML083460084), the applicant provided 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 provided the theoretical basis of the tsunami propagation used (Method of Splitting a Tsunami [MOST]) and its verification. However, the applicant did not thoroughly discuss the conservatism of input parameters. The applicant used 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 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.

<u>Conclusion:</u> In its response to RAI 02.04.06-1, the applicant and the 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 resolved and closed.

### 2.4S.6.4.2 Historical Tsunami Record

*<u>Information Submitted by Applicant</u>*

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).

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

The 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, "Probable Maximum Tsunami Hazards." 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: The 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 concludes the applicant's analysis acceptable.

Source Generator Characteristics

*Information Submitted by Applicant*

In FSAR Subsection 2.4S.6.3, the applicant stated 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.

*The 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.

<u>Intra-Plate Earthquakes</u>: 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.

<u>Inter-Plate Earthquakes</u>: 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 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).

<u>Local Submarine Landslides</u>: 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. The 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.* *East Breaks Landslide*

*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 $km^3$<br>Min: 20.80 $km^3$ | 519.52 $km^2$<br>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.* *Mississippi Canyon*

*Geologic Setting*: River delta and fan system.

*Age*: 7,500 to 11,000 years.

Maximum Single Event

| 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,<br>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 or 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 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.

*The 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 its response to RAI 02.04.06-1, , the applicant modeled a tsunami from the East Breaks landslide using a NSW wave model (MOST) that is described in FSAR Version 3.0. In contrast, the 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.

*The 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, the staff re-computed the offshore tsunami water levels for the northern Caribbean subduction zone and the northern South American convergent zone earthquake scenarios of ten Brink et al., (2008). These scenarios use the COMCOT model 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 concludes 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 concludes 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.

East Breaks Landslide

*1HD Results*: 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) (see SER Figure 2.4S.6.4.5-1). 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.

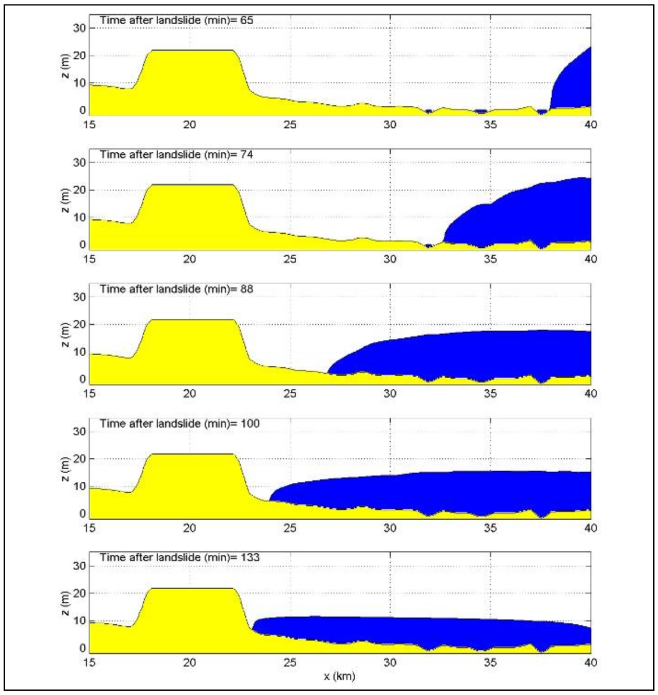


**Figure 2.4S.6.4.5-1 The Onshore Evolution of the 1HD Tsunami from the East Breaks Scenario for the Mid-Friction Case (Case "b") (A Cross-Sectional Profile of the Main Cooling Reservoir is Shown on the Left Side)**

*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, the 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.

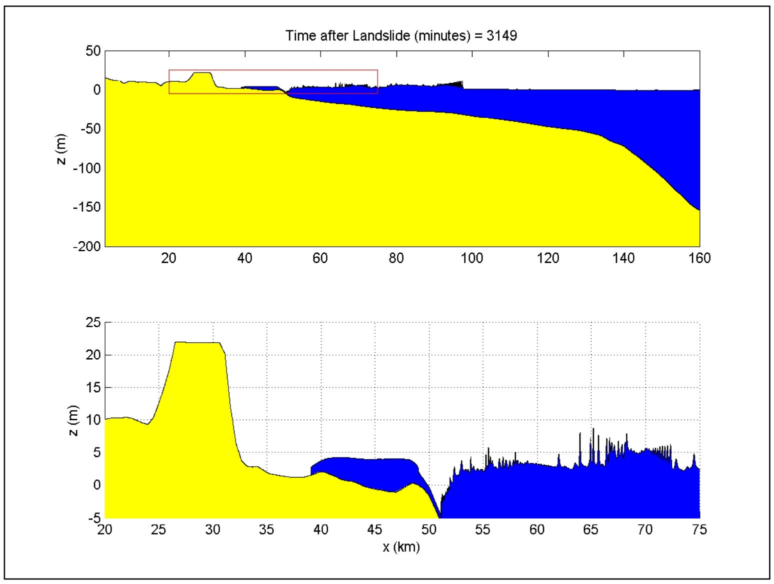


**Figure 2.4S.6.4.5-2 Wave Profile at the Time of Maximum Inundation for the Campeche 2HD 60-km Slide Width Source Scenario and for the Mid-Friction Case (Case "b") (Top) View Across the Continental Shelf (Bottom) View Near the STP Site**

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 its 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 concluded that there will be no onsite tsunami waves affecting safety-related facilities.

*The Staff's Technical Evaluation*

The 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***

The 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.

The 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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 stated 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 concluded 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 °C to 33.4 ºC (51.1 °F to 92.1 ºF) based on measurements between 1997, and 2005. Based on these data, the applicant concluded that there is no risk of ice formation near the STP site.

*The 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 five 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 stated 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 noted 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.

*The 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 stated 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 DBA without the storage basin receiving any makeup water.

The applicant stated 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.

*The 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 stated 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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, "Aerial View of the Site"). 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 stated 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 stated 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).

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

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

*<u>Information Submitted by Applicant</u>*

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 stated 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.

New CWS Intake and Discharge Structures

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 stated 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 estimated 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 stated 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 two-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 concluded 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 concluded that the energy of the PMH-generated waves will dissipate due to frictional losses and the raised water surface will decrease after each oscillation.

*The 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, "Section "A" - Unit 3 Rev. D," through 2.5S.4-49D, "Section "D" Rev. D"). The two UHS water-storage tanks, one for each proposed unit, will be located south of the respective units. Section 9.2.5, "Ultimate Heat Sink," of the FSAR evaluates the capacity of these UHS water-storage tanks. The staff determined that the these UHS water-storage tanks will be sufficient to meet 30 days of the UHS cooling requirements under DBA 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 its response to RAI 02.04.08-1, dated August 27, 2008 (ML091910403), the applicant stated 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 two-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 two-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 two-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 stated 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 stated 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 stated that FSAR Figures 2.4S.8-2 through 2.4S.8-5 will be deleted. The staff confirmed that in FSAR Revision 7, these figures are deleted. Therefore, RAI 02.04.08-1 is resolved and 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, "Aerial View of the Site," 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 its response to RAI 02.04.08-2, dated August 27, 2008 (ML091910403), the applicant stated 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 RAIs 02.04.08-1 and 02.04.08-2 are resolved and closed.

*Seiche in Main Cooling Reservoir*

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 finds 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 finds 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 finds 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.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, “Channel Diversions,” of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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

*Information Submitted by Applicant*

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 concluded 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 states 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 stated 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, "Human-Induced Changes of Channel Diversion"). However, the applicant stated 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 concluded that there is little likelihood of major channel diversions affecting STP, Units 3 and 4, safety facilities (FSAR Subsection 2.4S.9.5).

*The Staff's Technical Evaluation*

In its response to RAI 02.04.09-1, dated July 2, 2008 (ML081890239), the applicant stated that the flood of 1935, had a peak discharge of almost 14,158 $m^3/s$ (500,000 cfs). The applicant also stated that the 193,5 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 argued 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 concluded that the applicant's description of historical channel diversions is acceptable. Therefore, RAI 02.04.09-1 is resolved and closed.

2.4S.9.4.2 Stratigraphic Evidence

*Information Submitted by Applicant*

The applicant stated 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).

*The 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.

*The 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 concluded 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 concluded that the gravel mining effect will not contribute significantly to channel diversion near the STP site.

*The 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 platform. 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.

*The 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 concluded 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.

*The 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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, "COL License Information."

*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, "ABWR Site Parameters."

*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, "Flooding Protection Requirements," 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 RGs 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.

*Tier 1 Departure*

- 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 stated 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 stated 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 stated 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 stated that all safety-related facilities in the power block area are watertight below 12.2 m (40 ft) MSL. The applicant stated 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.

In its response to RAI 02.04.05-11, dated November 22, 2010 (ML103330369), the applicant describee the erosion protection features of the main cooling reservoir northern embankment. The applicant stated 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 stated 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 concluded 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 stated 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 stated 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 stated 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 stated 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 noted 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 its response to RAI 02.04.05-11 part (8), the applicant stated that water will flow past the main cooling reservoir northern embankment under the conservative PMSS scenario predicted by the SLOSH. The applicant noted 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 its response to RAI 02.04.05-11 part (9), the applicant stated 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 noted 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 concluded that any erosion at the base of the main cooling reservoir northern embankment would not threaten a failure.

In its response to RAI 02.04.05-11 part (10), the applicant stated 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 its response to RAI 02.04.05-11 part (11), the applicant stated 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 stated 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 concluded that erosion of the main cooling reservoir northern embankment is unlikely.

*The 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."

In its response to RAI 02.04.10-1, dated November 13, 2008 (ML083250480), the applicant stated 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 concludes this response reasonable acceptable and RAI 02.04.10-1 is resolved and closed.

The applicant reported 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 stated 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 seven hours in both the West and the East Channels. The applicant also stated that the duration of discharges exceeding 28.3 $m^3$/s (1,000 cfs) was less (one hour in both the West and the East Channels). The applicant noted that the local PMP event is a slow-moving event that allows the plant operators sufficient time to take action.

In its letter dated January 28, 2009 (ML091880126), the applicant reported 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 reported 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 stated 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 stated 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 resolved and closed.

#### *2.4S.10.5 Post Combined License Activities*

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

### *2.4S.10.6 Conclusion*

The 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 finds 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.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 ensured 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 of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant addressed the impacts of low water on safety-related water supply. In this section, the applicant provided 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 RGs 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 addressed 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 its response to RAI 02.04.11-1, dated June 26, 2008 (ML081970231), the applicant stated 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 resolved and 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 1967, 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.

*The 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 DBA 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 proposed 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 concluded that low water resulting from ice-induced causes is unlikely.

*The 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 DBA 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.

*The 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.

*The 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 DBA 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.

*The 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 DBA 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 DBA for 30 days, without any reliance on makeup water.

*The 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 DBA 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 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 DBA 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 found, 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 finds 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 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, and
- 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, "Correlation of USGS and Texas Nomenclature (modified from Reference 2.4S.12-2)," 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 described 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.

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

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

*<u>Information Submitted by Applicant</u>*

In FSAR Subsection 2.4S.12.1.4, and the applicant's proposed revision in its response to RAI 02.04.12-28, dated November 23, 2009 (ML093310392), the applicant described 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 noted that the Shallow Aquifer can be subdivided into an Upper Shallow Aquifer and a Lower Shallow Aquifer. The applicant stated 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 2008, 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 stated 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, "Representative Properties of Hydrogeologic Units," 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 (0.0005-0.05) | Table 2.4S.12-13 |
| 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 Surface 10 to 15 ft BGS (cont'd) | 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 |
| | 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 |

| Hydrogeologic Unit | Property | Units | Representative Value* | Range | FSAR Source |
|---|---|---|---|---|---|
| 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.<br>gm = geometric mean; am = arithmetic mean; pj = professional judgment; hyd = hydraulic;<br>hyd cond = hydraulic conductivity; pcf = pounds per cubic foot. | | | | | |

*The 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 STP, Units 1 and 2 UFSAR Revision 13, Figures 2.4.13-17and 2.4.13-17a, "Ground Water Contour Map Lower Shallow Aquifer Zone" (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 staff confirmed that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-28 is resolved and 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 described 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.

*The 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 its response to RAI 02.04.12-28, dated September 21, 2009 (ML092710096), the applicant described 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.

*The 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.

In its 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), the applicant provided 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 stated 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 staff confirmed that the applicant has incorporated the proposed changes in the FSAR. Therefore, RAI 02.04.12-36 is resolved and 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.

**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 | | | |

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.

*The 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. In its response to RAI 02.04.12-04, dated July 2, 2008 (ML081890239), the applicant 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).

*The 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 STP, 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 STP, Unit 3, shows a peak groundwater elevation of 8.38 m (27.49) ft MSL. Observation well 993U, between proposed STP, 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.

*The 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.

In response to RAI 02.04.12-28, dated August 5, 2009 (ML092170354), it is requested 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 STP, Units 1 and 2. The staff concludes that a similar response to excavation and fill at the proposed location of STP, Units 3 and 4, can be anticipated. Post-construction simulations of STP, 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 concluded 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 states 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 reported 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, "Representative Properties of Hydrogeologic Units"). 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.

*The Staff's Technical Evaluation*

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, the staff's responses are located in 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 noted 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.

*The 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 resolved and 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 STP, 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 STP, 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.

*The 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.

In its response to RAI 02.04.12-17, dated August 12, 2008 (ML082270381), the applicant 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. Therefore, RAIs 02.04.12-17, 02.04.12-29, and 02.04.12-30, are resolved and 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 stated 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 resolved and 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 STP, Units 3 and 4, because construction of the proposed units requires a similar excavation and fill. Therefore, RAI 02.04.12-42 is resolved and 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 STP, 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 STP, 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 STP, 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 STP, 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 STP, 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**

*Information Submitted by Applicant*

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, "Estimated Average Linear Velocity and Travel Time," presents travel times for the four pathways analyzed. The applicant revised and presented this table in its 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 | Average 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 | | | | |

*The 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 its 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 its 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. Therefore, RAIs 02.04.12-28 and 02.04.12-38 are resolved and closed 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 STP, 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 $2.0 \times 10^{-2}$ 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 STP, Units 3 and 4, a slurry wall surrounding Units 3 and 4, and postulated releases from STP, Units 3 and 4. Model results provide projections of groundwater hydraulic head and pathways from releases to receptor points.

In its 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 STP, 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.

*The 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 2010, 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.

In its response to RAI 02.04.12-47 (ML102450252), dated August 30, 2010, the applicant addressed 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 concluded 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 resolved and closed.

The applicant's 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 stated 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 STP, 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 STP, 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).

*The 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, and
- 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 resolved and closed.

The staff noted that the applicant's response to RAI 02.04.12-24, stated 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 stated 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 stated 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, and 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 STP, 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 STP, 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 concluded 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. In its response to RAI 02.04.12-26, dated July 24, 2008, the applicant 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 concluded 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 assumed that this same concept will apply to STP, Units 3 and 4.

*The 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 its response to RAI 02.04.12-51 (ML103330369), dated November 22. 2010, the applicant noted that minor excavations into the clay cap could occur over the life of the plant. The applicant also noted 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 in its response to RAI 02.04.12-51, dated November 22, 2010, the staff 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. The staff issued RAI 02.04.12-51, 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 the staff's review of the 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 resolved and closed.

The applicant described 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 resolved and 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 resolved and 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, finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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.

*The 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 concluded 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 resolved and 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 m3 (36,984 gal) (ABWR DCD Tier 2, Section 11.2, Table 11.2-4, "Capacities of Tanks, Pumps, and Other Components"). 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, "Low

Conductivity Waste Collection Tank and Reactor Coolant Radionuclide Inventory," 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 noted 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.

*The 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 Branch Technical Position (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, "Source Terms").

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 concluded 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 STP, 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, and
- 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 concluded 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 STP, Unit 4, to the western site boundary is also assumed to be intercepted by a water-supply well.

*The 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, and
- 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 STP, 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

<u>*Information Submitted by Applicant*</u>

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, "Decay Chains Considered in Accidental Effluent Release," 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 one 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 STP, 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 STP, 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.

*The 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, "Screening Analysis Considering Radioactive Decay and Representative Conditions," and 2.4S.13-2B, "Screening Analysis Considering Radioactive Decay and Fastest Flow Conditions" (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. In its response to RAI 02.04.13-8 (ML082270381), dated August 12, 2008, the applicant stated 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 resolved and 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 STP, 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 one for each pathway.

The applicant also performs a sensitivity analysis using the range of average linear velocity (see FSAR Table 2.4S.12-17, "Estimated Average Linear Velocity and Travel Time"; 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 noted 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 concluded that the STP site groundwater pathway yielded an analysis that demonstrated compliance with 10 CFR Part 20, Appendix B, Table 2, "Effluent Concentrations." Compliance was demonstrated for both individual radioisotopes and through the sum of fractions, for mixtures of radioisotopes.

*The 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 noted 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 its response to RAI 02.04.13-13, dated September 16, 2009 (ML092610376), the applicant used 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.

*The 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 finds 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.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 of the STP, Units 3 and 4, COL FSAR Revision 12, 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 RGs 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 stated 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 stated 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 stated 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 concluded that no EOPs or plant TS are required to implement flood protection for STP, Units 3 and 4.

The applicant stated 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 concluded that adequate time is available to implement remedial or preventive measures for non-safety-related facilities.

The applicant stated 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, “Section “A” - Unit 3 Rev. D,” through 2.5S.4-49D, “Section “D” Rev. D”). 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 DBA conditions, without needing any makeup or blowdown.

*The 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.

In its response to RAI 02.04.14-1, dated January 28, 2009 (ML090300648), the applicant provided 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 stated 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 stated 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 resolved and 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 finds 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).

## 2.5S Geology, Seismology, and Geotechnical Engineering

Section 2.5S, of the FSAR describes geologic, seismic and geotechnical engineering properties of the proposed STP COL application site. FSAR Section 2.5S.1, "Basic Geologic and Seismic Information," discusses geologic and seismic characteristics of the COL site and the region surrounding the site. FSAR Section 2.5S.2, "Vibratory Ground Motion," describes the vibratory ground motion assessment for the COL site through a probabilistic seismic hazard analysis (PSHA) and develops the site-specific, safe-shutdown earthquake (SSE) ground motion. FSAR Section 2.5S.3, "Surface Faulting," evaluates the potential for surface tectonic and non-tectonic deformation at the COL site. FSAR Section 2.5S.4, "Stability of Subsurface Materials and Foundations," and FSAR Section 2.5S.5, "Stability of Slopes," describe foundation and subsurface material stability at the COL site.

Following NRC guidance in RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR-Edition)," and in RG 1.208, "A Performance-Based Approach to Define Site-Specific Earthquake Ground Motion," the applicant defined the following four zones around the STP site and conducted investigations within those zones:

- Site region – Area within 320 kilometers (km) (200 miles [mi]) of the site location.
- Site vicinity – Area within 40 km (25 mi) of the site location.
- Site area – Area within 8 km (5 mi) of the site location.
- Site location – Area within 1 km (0.6 mi) of the proposed STP, Units 3 and 4.

The applicant used the previous site investigations for STP, Units 1 and 2, (located adjacent to the proposed STP, Units 3 and 4) as a starting point for the characterization of the geologic, seismic, and geotechnical engineering properties of the COL site. As such, the material in Section 2.5S of the COL application focuses on new information published since the issuance of the STP, Units 1 and 2, Updated Final Safety Analysis Report (UFSAR). The material in FSAR Section 2.5S of the COL application also focuses on any recent geologic, seismic, geophysical, and geotechnical investigations performed for the COL site.

The applicant used seismic source and ground motion models published by the Electric Power Research Institute (EPRI) in "Seismic Hazard Methodology for the Central and Eastern United States (CEUS), Tectonic Interpretations," (EPRI, 1986) as the starting point for characterizing potential regional seismic sources and the resulting vibratory ground motion. The applicant then updated these EPRI seismic source and ground motion models or incorporated new data into the PSHA, in light of more recent data and evolving knowledge. For the STP site, the applicant incorporated Rio Grande faults associated with the Rio Grande Fault Zone and a revised New Madrid seismic zone (NMSZ) source zone into its PSHA. The applicant employed the performance-based approach described in RG 1.208 to develop the Ground Motion Response Spectra (GMRS) for the site. In addition, consistent with SECY-12-0025, "Proposed Orders and Requests for Information in Response to Lessons Learned from Japan's March 11, 2011, Great Tohoku Earthquake and Tsunami," as well as with the need to consider the latest available information in the PSHA for the STP site specified in RG 1.208, the applicant performed a sensitivity study using the central and eastern United States seismic source characterization (CEUS-SSC) model presented in NUREG-2115, "Central and Eastern United States Seismic Source Characterization for Nuclear Facilities." The applicant's sensitivity study showed that the CEUS-SSC GMRS for the STP site is very close to, and not significantly above, the STP COL application FSAR GMRS, while the STP site-specific SSE is above both GMRSs. Based on the results of its sensitivity study, the applicant concluded that the STP COL FSAR seismic design basis did not need to be revised. SER Section 22.1 presents the staff's evaluation of the applicant's sensitivity study.

This SER, written by staff, is divided into five main parts (SER Sections 2.5S.1 through 2.5S.5) that parallel the five FSAR sections prepared by the applicant for the STP COL application. The discussion that follows presents the staff's safety evaluation of the geology, seismology, and geotechnical engineering for the proposed STP, Units 3 and 4.

## 2.5S.1 Basic Geologic and Seismic Information

### *2.5S.1.1 Introduction*

FSAR Section 2.5S.1 of the STP COL application includes geologic information that the applicant collected during regional and site investigations. This technical information results primarily from surface and subsurface geologic, seismic, geophysical, and geotechnical investigations performed in progressively greater detail closer to the site and within each of four circumscribed areas corresponding to site region, site vicinity, site area, and site location, as previously defined. The primary purposes for conducting these investigations are to determine the geologic and seismic suitability of the site, to provide the bases for the plant design, and to determine whether there is significant new tectonic or ground motion information that could impact the seismic design bases, as determined by a PSHA. The applicant's basic geologic and seismic information in FSAR Section 2.5S.1 addresses the regional and site geology, tectonic and seismic characteristics, non-tectonic deformation, and conditions caused by human activities.

### *2.5S.1.2 Summary of Application*

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52 Appendix A. The applicant identifies no departures from the certified design and provides supplemental, site-specific information to address COL License Information Item 2.23.

*COL License Information Item*

- COL License Information Item 2.23 Geology, Seismology, and Geotechnical Engineering

This COL license information item provides site-specific information related to the regional and site geologic, seismic, and geophysical conditions, including those caused by human activities.

FSAR Section 2.5S.1 of the STP, Units 3 and 4, COL FSAR Revision 12, describes the geologic and seismic characteristics of the STP site region and site area. FSAR Subsection 2.5S.1.1, “Regional Geology,” describes the geologic and tectonic setting within a 320-km (200-mi) radius of the site, while FSAR Subsection 2.5S.1.2, “Site Area Geology,” describes the geology and tectonic setting of the 40-km (25-mi), 8-km (5-mi), and 1-km (0.6-mi) site radii.

The applicant developed FSAR Section 2.5S.1 after reviewing the relevant published geologic literature; conducting geologic field investigations; and interviewing experts in the geology, seismology, and tectonics of the site region. The applicant’s field investigations include geologic field and aerial reconnaissance, subsurface geophysical and geotechnical investigations, and aerial photographic and remote sensing imagery analyses. In addition, the applicant uses the previous UFSAR (South Texas Project Electric Generating Station [STPEGS], 2006) for the existing STP, Units 1 and 2, to supplement its recent geologic investigations of the site.

The applicant applied the information in FSAR Section 2.5S.1, toward developing a basis for evaluating the geologic and seismic hazards discussed in succeeding sections of the FSAR. Based on this evaluation, the applicant presents the following information related to the regional and site geology for the STP COL site.

2.5S.1.2.1 Regional Geology

FSAR Subsection 2.5S.1.1, describes the regional physiography, geomorphology, geologic history, stratigraphy, and tectonic setting within a 320-km (200-mi) radius of the STP COL site. The following SER sections summarize the applicant’s information in FSAR Subsection 2.5S.1.1.

**Regional Physiography and Geomorphology**

FSAR Subsection 2.5S.1.1.1, discusses the regional physiography and geomorphology surrounding the STP site. The applicant stated that the site is located within the Coastal Prairie subsection of the Gulf Coastal Plain Physiographic Province. SER Figure 2.5S.1-1 (reproduced from FSAR Figure 2.5S.1-1, “Map of Physiographic Provinces”) illustrates the location of the STP site with respect to the Coastal Plain Province and neighboring physiographic provinces, the Texas-Louisiana Shelf, the Texas-Louisiana Slope, and the Great Plains section of the Edwards Plateau Province. FSAR Figure 2.5S.1-6, “Physiographic Map of Texas,” illustrates all of the physiographic provinces within the state of Texas and briefly describes each province and subprovince.

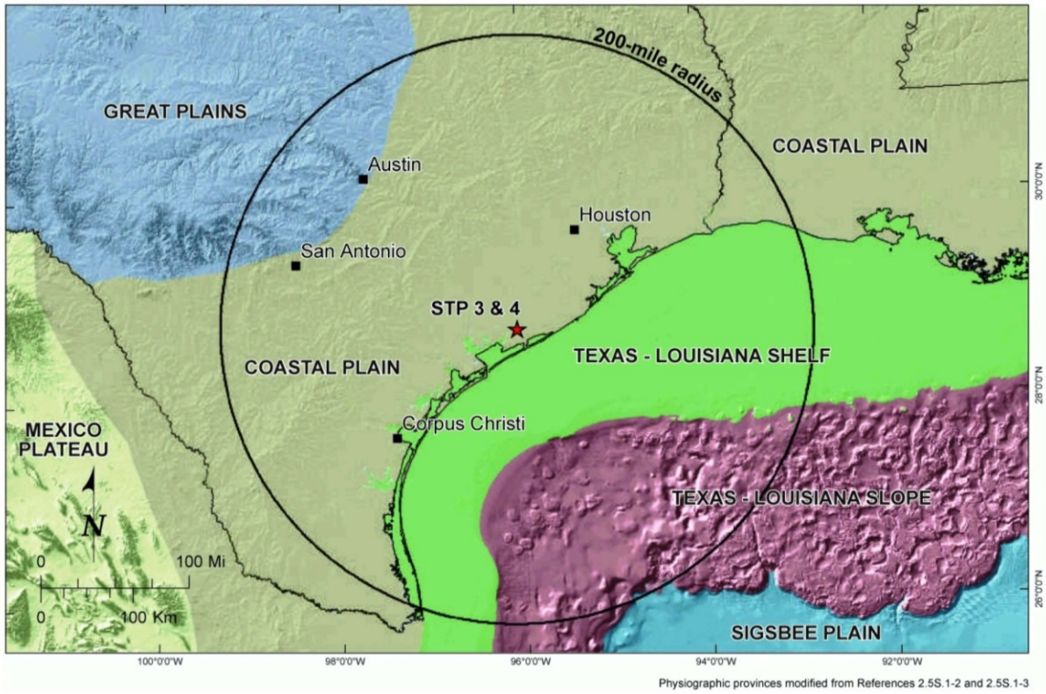


**Figure 2.5S.1-1 Map of Physiographic Provinces within the STP Site Region (FSAR Figure 2.5S.1-1)**

FSAR Subsection 2.5S.1.1.1.1, describes the Gulf Coastal Plain Physiographic Province, which is divided into three subprovinces (the Coastal Prairies, the Interior Coastal Plains, and the Blackland Prairie). Each subprovince lies within the western and northwestern portion of the site region. The Coastal Prairie subprovince, where the STP site is located, is the southeastern most portion of the Gulf Coastal Plain. The applicant describes the Coastal Prairie as nearly flat, extending approximately 80 to 120 km (50 to 75 mi) from the Gulf of Mexico, with a maximum difference in elevation of 91.5 meters (m) (300 feet [ft]). The elevation of the STP site is 9.1 m (30 ft) above mean sea level. The applicant also notes that the Coastal Prairie subprovince is characterized by thick Quaternary-age (less than 1.8 million years old [Ma]) unconsolidated deltaic sands and muds. The southeastern edge of the Coastal Prairies (and thus the Gulf Coastal Plain Province) follows the Gulf of Mexico coastline and marks the transition to the Texas-Louisiana Shelf, which is less than 40 km (25 mi) from the STP site.

In FSAR Subsection 2.5S.1.1.1.2, the applicant describes the Edwards Plateau as dominated by limestone and dolomite that contain both sinkholes and caverns. The southern and eastern edges of the Edwards Plateau are characterized by normal faults that are part of the Balcones Escarpment. The applicant discusses the Balcones Fault Zone in FSAR Subsection 2.5S.1.1.4.1.

FSAR Subsection 2.5S.1.1.1.3, describes the Texas-Louisiana Shelf Physiographic Province as a broad, nearly featureless plain that has spread seaward, or prograded, as the Gulf of Mexico waters have transgressed throughout the Cenozoic Era (65 Ma to the present). Finally, in FSAR Subsection 2.5S.1.1.1.4, the applicant describes the Texas-Louisiana Slope. This continental slope is characterized by uneven topography with a gradient that ranges between 1° and 20°. This variation is due to the presence of Jurassic-age (206 to 144 Ma) salt at a depth that has migrated upward, leading to the formation of mound-like features or knolls as well as basins. The Texas-Louisiana Slope is also characterized by growth faulting that occurs along the break from the Texas-Louisiana Shelf. Growth faulting is common in the gulf coastal region where sedimentary units have experienced rapid deposition.

**Regional Geologic History**

FSAR Subsection 2.5S.1.1.2, describes a complex geologic history that spans approximately 1 billion years and includes three orogenic (mountain building) events divided in time by two major extensional (rifting) events. The applicant stated that direct evidence for these events at the STP site is buried beneath approximately 12 km (40,000 ft) of unconsolidated sediments. According to FSAR Subsection 2.5S.1.1.2.4, before the Cretaceous Period (144 to 65 Ma), there was no connection between the present day Gulf of Mexico and the Atlantic Ocean. The applicant noted that this connection was established after Jurassic-age (206 to 144 Ma) rifting ended and the Gulf of Mexico Basin became tectonically stable, and transgressing seas covered the land bridge that once connected present day Florida and the Yucatan Platform.

FSAR Subsection 2.5S.1.1.2.6, describes the Cenozoic (65 Ma to the present) geologic history. The applicant explains that loading of the crust due to the rapid seaward deposition of sediments during the Cenozoic Era led to subsidence of the Gulf of Mexico Basin, which has continued through to the present. As a result of rapid sedimentation during Cenozoic time, growth faults developed throughout the coastal region. The STP site area experienced its most abundant sediment accumulation between 54.8 and 23.8 Ma, before the depositional center migrated southward and eastward. During the Quaternary Period (1.8 Ma to the present), periods of continental glaciations (and interglaciations) contributed to sequences of sea level rise (transgression) and fall (regression). These sequences are recorded in marine sedimentary deposits.

**Regional Stratigraphy**

FSAR Subsection 2.5S.1.1.3, describes the regional stratigraphy for the Coastal Plain Physiographic Province. The applicant stated that there is little subsurface boring data available to characterize pre-Cenozoic sediments associated with the Gulf of Mexico Basin. The thickness of the Cenozoic sediments masks the basement rock and the pre-Cenozoic sediments that make drilling beneath the Cenozoic sediments difficult at best. According to the applicant, outcrop exposures in the Llano Uplift (on the northwestern edge of the site region), the Marathon Uplift of west Texas, and the more distant Ouachita and Appalachian mountains provide the basis for what is known about the Paleozoic rock beneath the Coastal Plain Province. SER Figure 2.5S.1-2 (reproduced from FSAR Figure 2.5S.1-5, "Regional Geological Map (200-mile radius") is a geologic map of the STP site region showing the limited Paleozoic exposures (in purple).

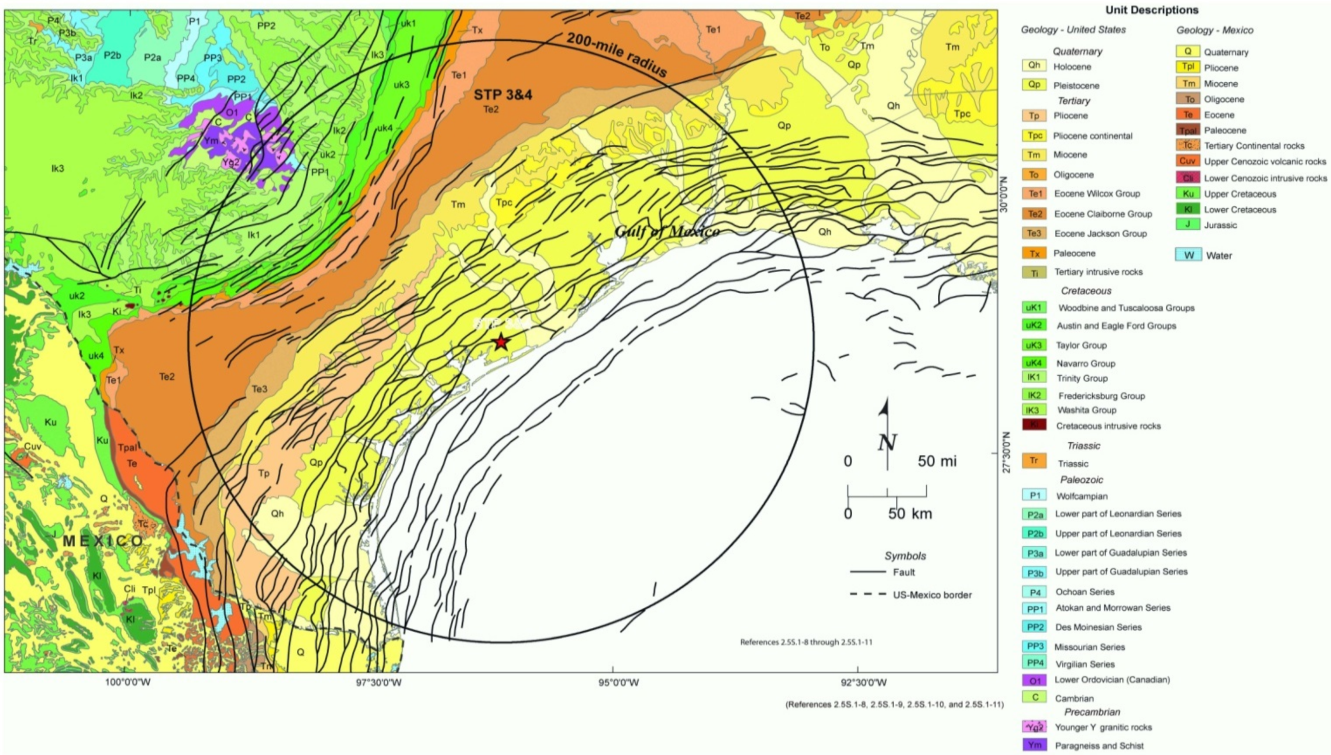


**Figure 2.5S.1-2 Geologic Map of the STP Site Region (FSAR Figure 2.5S.1-5)**

FSAR Subsection 2.5S.1.1.3.3, discusses the formation of the Gulf of Mexico Basin following the breakup of Pangea and the opening of the Atlantic Ocean during the Mesozoic Era. The applicant stated that rifting associated with the breakup began during the Triassic Period (248 to 206 Ma) and lasted into the Jurassic Period (206 to 144 Ma). There are no exposures of Triassic-age rock or sediments within the STP site region. The applicant discusses Jurassic age Louann salt deposits and interprets them to be present beneath the STP site region based on limited petroleum exploration borings. FSAR Figures 2.5S1-10, "Geologic Features of the Gulf Coast Region," and 2.5S.1-11, "Site Vicinity Geologic Map (25-mile radius"), illustrate salt migration structures that are present within the site region and site vicinity. The FSAR states that by the end of the Jurassic Period, sea level rose, marine waters transgressed (migrated) landward, and the Gulf of Mexico became connected to the Atlantic Ocean.

FSAR Subsection 2.5S.1.1.3.4, discusses the stratigraphic history of the Cretaceous Period (144 to 65 Ma). During this time, the Gulf of Mexico Basin was tectonically stable, but growth faulting was prominent along the coastal margin as a result of rapid accumulations of sediments from areas to the north and northwest. SER Figure 2.5S.1-2 illustrates the abundance of growth faults surrounding the STP site.

FSAR Subsection 2.5S.1.1.3.5, discusses Cenozoic-age (65 Ma to the present) stratigraphic sequences as they relate to the STP site region. The applicant provides a generalized stratigraphic column for the Cenozoic Era in FSAR Figure 2.5S.1-13, "General Cenozoic Stratigraphic Column." Based on this figure, 4,500 to 6,000 m (15,000 to 20,000 ft) of Cenozoic sediments are present beneath the site. The applicant stated that these thickness estimates are based mostly on oil field (petroleum) logs. Minor marine, fluvial, deltaic, and volcanic-derived sediments make up a majority of the thick Cenozoic deposits. The most recently deposited sedimentary units are from the Pleistocene Epoch (1.8 MA to 10,000 years ago). These deposits reflect cyclic sea level changes that coincide with four Pleistocene continental glaciations discussed in FSAR Subsections 2.5S.1.1.1.1 and 2.5S.1.1.2.6 and 2.5S.1.2. The applicant stated that the volume of material deposited during the Pleistocene Epoch led to the subsidence of the Gulf of Mexico Basin and subsequent growth faulting, as well as upward salt mobilization. The effects of growth faulting and salt mobilization are discussed further in FSAR Subsections 2.5S.1.2 and 2.5S.1.1.4.4.4 and Section.2.5S.3.

FSAR Subsection 2.5S.1.1.3.5.6, describes the Beaumont Formation, the upper Pleistocene stratigraphic unit that is composed of alluvial fan deposits. This unit underlies the STP site and based on FSAR Figure 2.5S.1-13, is approximately 122 m (400 ft) thick. The applicant explains that the actual thickness of the Beaumont is difficult to confine because the thickness is variable, and the composition is similar to that of the underlying lower Pleistocene Lissie Formation. FSAR Subsection 2.5S.1.1.4.1.3, states that the Beaumont Formation was deposited during the Sangamon interglacial episode approximately 120 thousand years ago (Ka), when sea level was high and the Gulf of Mexico shoreline was migrating southward. As the shoreline retreated to its present location during the most recent Wisconsinan glaciation, the Beaumont deposits were subject to weathering and erosion.

**Regional Tectonic Setting**

FSAR Subsection 2.5S.1.1.4 describes the regional tectonic setting for the STP COL site. The applicant stated that the site is located in the stable continental region (SCR) of the CEUS. The applicant also states that the 1986 EPRI study, a regional study that defines seismic source models for the CEUS, includes the STP site region. In FSAR Subsection 2.5S.1.1.4, the applicant discusses the regional: (1) tectonic history, (2) tectonic stress, (3) gravity and magnetic data, and (4) tectonic structures based on the "current state of knowledge" and information that post-dates the 1986 EPRI study. The applicant concluded that: (1) there is no evidence for late Cenozoic (Quaternary) (1.8 MA to the present) seismic activity on any known geologic structure, and (2) there is no information available that would require an update to the 1986 EPRI source models (EPRI, 1986) for the site region (within a 320-km [200-mi] radius of the site).

*Regional Tectonic History*

In FSAR Subsection 2.5S.1.1.4.1, the applicant stated that continental-scale collisional (mountain-building) events during the late Paleozoic Era (354 to 248 Ma) largely influenced the geologic structure of the crust beneath the STP site region. During this mountain-building episode known as the Ouachita orogeny, the ancestral continents of Africa and North America (Laurentia) collided with one another. The Gulf Coastal Plain Physiographic Province, where the STP site is located, formed during the subsequent opening of the Gulf of Mexico Basin during the Mesozoic Era (248 to 65 Ma). The applicant explains that this basin formed along the trend of the Ouachita orogenic belt and, within the site region, remnants of the orogenic belt are buried beneath Mesozoic and Cenozoic stratigraphic units.

Based on interpretations of gravity data, the most significant evidence for Mesozoic rifting and extension associated with the formation of the Gulf of Mexico is located beneath the present continental shelf. However, this rifting episode affected all of the crust within the site region. In FSAR Subsection 2.5S.1.1.4.1.3, the applicant identifies four types of crust that are present within the 320-km (200-mi) site radius: (1) extended continental crust, (2) extended thick transitional crust, (3) extended thin transitional crust, and (4) Mesozoic oceanic crust. The STP site lies within the thin transitional crustal zone, an area that may have experienced greater thinning due to "locally elevated crustal temperatures."

Sedimentary deposition following Mesozoic rifting and continued sedimentary loading through recent geologic times led to: (1) the accumulation of approximately 12 km (40,000 ft) of Mesozoic and Cenozoic sediments, (2) subsidence of the thin transitional crust, and (3) the formation of salt diapers and growth faults within the Gulf Coastal Plain. Growth faults are non-tectonic normal faults common throughout the Gulf of Mexico region. The applicant stated that the U.S. Geological Survey (USGS) considers the Gulf of Mexico growth faults to be "Class B" structures based on the fact that these faults do not penetrate crystalline basement rocks and are therefore less likely to initiate "significant earthquakes" (Wheeler, 2005). The applicant discusses growth faults in greater detail in FSAR Subsections 2.5S.1.1.4.4.5.4 and 2.5S.1.2.4.2 and Section 2.5S.3.

*Regional Tectonic Stress*

FSAR Subsection 2.5S.1.1.4.2, discusses regional tectonic stresses acting on the CEUS as well as localized stresses present in the STP site region. The tectonic stress in the CEUS, including the gulf coastal region, is primarily characterized by northeast and southwest-directed horizontal compression. This compression is due to ridge-push force from the mid-Atlantic ridge, which is transmitted to the interior of the North American tectonic plate. However, the applicant stated that there are additional localized stresses that influence the STP site region. For example, the site region may be locally influenced by the flexural loading of the crust due to significant sedimentary deposition. In addition, buoyancy forces due to uplift in the Basin and Range to the west of the site may also account for localized perturbations in the stress field. The applicant stated that: (1) information reported since the 1986 EPRI study (EPRI, 1986) supports the initial EPRI findings, and (2) there is no significant change in the understanding of tectonic stress in the CEUS or the Gulf Coastal Plain. Therefore, the applicant concluded that it is not necessary to reevaluate the seismic potential of tectonic sources in the region with respect to the regional tectonic stress field.

*Regional Gravity and Magnetic Data*

FSAR Subsection 2.5S.1.1.4.3, discusses regional gravity and magnetic data in relation to the STP site region. The applicant reviewed data sets with scales (grid spacing) that allow for the identification and assessment of gravity and magnetic anomalies with wavelengths tens of miles or greater. The applicant relies primarily, but not solely, on the published gravity data sets from the Geological Society of America (NOAA-NGDC, 1999) and on the magnetic anomaly data of Bankey et al. (USGS, 2002a, 2002b) and Keller (GSA, 1989). FSAR Figure 2.5S.1-22 shows gravity and magnetic anomaly profiles oriented northwest-southeast across the STP site region or along a 640-km (400-mi) transect. The applicant stated that the gravity and magnetic anomalies identified in the data represent the following three major tectonic events discussed in the FSAR: (1) Precambrian-Cambrian rifting, (2) the Paleozoic Ouachita orogeny, and (3) the opening of the Gulf of Mexico during the Mesozoic Era. The applicant discusses long-wavelength gravity highs and lows that correspond to the depth of basement rock. In addition, the applicant describes ten individual gravity features (features A through J) and six magnetic features (features A through F) that were identified from gravity and magnetic anomaly maps. These features are shown in FSAR Figures 2.5S.1-15, "Gravity Anomaly Features in Site

Region (200-mile radius)," and 2.5S.1-16, "Magnetic Anomaly Features in Site Region (200-mile radius)." The applicant does not suggest that any of these gravity or magnetic features represent structures that were unknown at the time of the 1986 EPRI study.

*<u>Principal Tectonic Structures</u>*

FSAR Subsection 2.5S.1.1.4.4, discusses principal regional tectonic structures in the STP site region based on information published since the 1986 EPRI study. SER Figure 2.5S.1-3 (reproduced from FSAR Figure 2.5S.1-17) shows all of the known tectonic features in the STP site region. The applicant concluded that none of this more recent information justifies a "significant change in the EPRI seismic source model."

The applicant categorizes the regional tectonic structures based on the age of formation or the most recent tectonic activity and states that Late Proterozoic, Paleozoic, and Mesozoic structures relate to major tectonic events, while Cenozoic (Tertiary and Quaternary) structures reflect the tectonic conditions within the Gulf of Mexico passive margin. The applicant does not discuss any Late Proterozoic structures within the STP site region because they are not exposed at the surface and are not well-constrained by data. The applicant discusses the following Paleozoic tectonic structures: (1) the Luling Thrust (or Luling Front), (2) the Kerr Basin, and (3) the Fort Worth Basin. These features are not exposed at the surface, and the two foreland basins are outside of the site region. Furthermore, the applicant presents no evidence to suggest that these features have been active since the Paleozoic Era.

<u>Mesozoic Tectonic Structures</u>

FSAR Subsection 2.5S.1.1.4.4.3, briefly discusses the implications of Mesozoic faulting (normal and transform faults) of basement rock due to extension and rifting. However, the existence and location of such features within the STP site region are not understood, and no seismicity data suggest the presence of such features. The interpretation that these faults are present within the regional thick and thin transitional crust is based on combining multiple data sets that include gravity, magnetic, and seismic data.

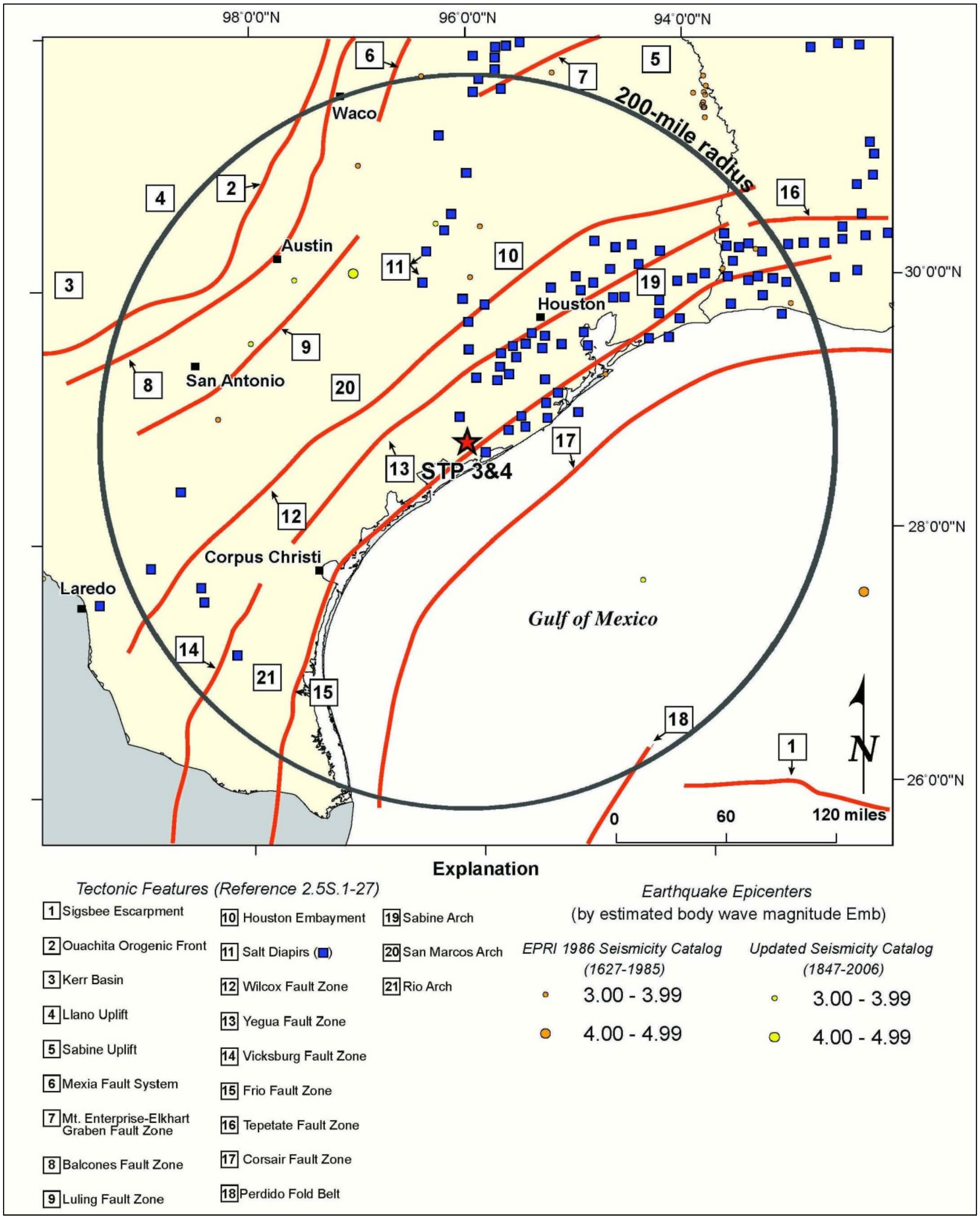


**Figure 2.5S.1-3. Tectonic Figures in the STP Site Region (FSAR Figure 2.5S.1-17)**

The applicant identifies the following Mesozoic fault systems that are interpreted to be related to the movement of buried Jurassic salt deposits: (1) the Mexia-Talco Fault System (including the Milano Fault Zone) in the northeastern portion of the site region; (2) the Charlotte-Jourdanton Fault Zone (including the Karnes Fault Zone) to the west of the STP site; and (3) the Mt.

Enterprise-Elkhart Graben (MEEG) system that barely extends into the northern portion of the site region. The applicant stated that there is evidence: (a) for movement on each of these fault systems between the Jurassic and early Tertiary times (before about 50 Ma), and (b) that each system is rooted in Jurassic Louann Salt deposits, not in crystalline basement rock. The applicant further states that there is some evidence for late Quaternary deformation on the MEEG. Therefore, the applicant provides additional details about this fault zone in FSAR Subsection 2.5S.1.1.4.4.5.1.

<u>Tertiary Tectonic Structures</u>

FSAR Subsection 2.5S.1.1.4.4.4, discusses early Cenozoic (or Tertiary [65 to 1.8 Ma]) salt structures, growth faults, and "basement-involved" faults. The applicant indicates that sedimentary processes dominated during the Tertiary Period and that major tectonic events did not impact the STP site region. Several processes led to the development of salt structures and growth faults in the gulf coastal region, including: (1) the continuous deposition of sediment, (2) gulfward migration of the shoreline, (3) loading and compaction of sedimentary strata, (4) flexure of the crust due to loading, and (5) gravitational gulfward slumping. The following sections describe these sedimentary features as well as two tectonic fault zones, the Luling and the Balcones, both of which experienced displacement during the Tertiary Period.

*Tertiary Salt Structures*

FSAR Subsection 2.5S.1.1.4.4.4.1, identifies three primary salt diaper provinces in the STP site region. Of these three, the Houston Diaper Province is closest to the STP site. The applicant noted that the UFSAR for existing STP, Units 1 and 2, describes three salt domes (Big Hill, Hawkinsville, and Markham) that are located within this province and within 24 km (15 mi) of the STP site. The applicant stated that no additional "large-scale" salt structures are known to exist within the 40-km (25-mile) STP site vicinity.

*Tertiary Growth Faults*

FSAR Subsection 2.5S.1.1.4.4.4.2, describes five growth fault zones within the STP site region that trend east-northeast to south-southwest, which is coincident with the trend of the Gulf of Mexico shoreline. The applicant indicates that as the shoreline migrated toward the gulf since the late Mesozoic, the active growth fault zones also migrated. The applicant noted that none of the growth fault zones within the site region penetrates into the crystalline basement rock. Instead, these zones all merge into detachment horizons (weak stratigraphic layers) of salt and/or shale. The five growth fault zones from north to south and by descending age are: (1) the Wilcox Fault Zone, (2) the Yegua Fault Zone, (3) the Vicksburg Fault Zone, (4) the Frio Fault Zone, and (5) the Corsair Fault Zone. The late Oligocene (approximately 23.8 Ma) Frio Fault Zone is located closest to the STP site. The applicant stated that the Frio is approximately 60 km (37 mi) in width and terminates against a deep detachment horizon. The Corsair Fault Zone is an offshore growth fault complex located southeast of the STP site that formed during the middle Miocene Epoch (approximately 15 Ma). The applicant assesses the potential for more geologically recent deformations associated with growth faults in FSAR Subsections 2.5S.1.1.4.4.5.4 and 2.5S.1.2.4.2.

*Tertiary Basement Involved Faults*

In FSAR Subsection 2.5S.1.1.4.4.4.3, the applicant discusses the Balcones and Luling Fault Zones that are located northwest of the STP site, within the site region. Both of these fault zones follow the northeast-southwest trend of the buried Ouachita Orogenic Belt and exhibit normal fault displacements of greater than 300 m (1,000 ft). The Balcones faults dip to the southeast and the Luling faults dip to the northwest, forming a graben system. SER Figure 2.5S.1-3 shows the Balcones and Luling Fault Zones in relation to the STP site. The applicant presents varied explanations for how these structures may have formed. One explanation suggests that the faults are controlled by pre-existing thrust faults at depth that originally formed during the Ouachita Orogeny and were then reactivated as normal faults during the Tertiary Period. This explanation is based mostly on deep seismic reflection data. Another explanation suggests that these faults formed during the Miocene and reflect tensile forces above a hingeline that was created in response to sedimentary loading and crustal flexure. The applicant does not favor one explanation over another. FSAR Subsection 2.5S.1.1.4.4.5.2 includes additional details on the Balcones Fault Zone.

*Quaternary Tectonic Structures*

In FSAR Subsection 2.5S.1.1.4.4.5, the applicant stated that neither the 1986 EPRI seismic source model studies (EPRI 1986) or the investigations for STP, Units 1 and 2, identify any capable tectonic structures within the STP site region. Nevertheless, the applicant discusses three features that exhibit potential Quaternary displacement, two of which (the MEEG and the Balcones Fault Zone) lie within the 320-km (200-mi) STP site radius. The third feature, the NMSZ, is located more than 800 km (500 mi) from the STP site. Although the applicant does not conclude that the NMSZ contributes significantly to the hazard at the site, this source zone has produced large historical earthquakes, and new information is available regarding the source zone parameters. The applicant discusses the updated seismic source characterization for the NMSZ in FSAR Subsection 2.5S.2.4.4.2.

FSAR Subsection 2.5S.1.1.4.4.5, discusses Quaternary growth faulting within the STP site region, even though growth faults are considered non-tectonic structures. The applicant concluded that there is no new information regarding Quaternary activity associated with any growth fault features that requires a revision (i.e., update) of the EPRI seismic source characterization of the coastal plain region.

*Mt. Enterprise-Elkhart Graben (MEEG) System*

The applicant stated that data indicating Quaternary age deformation and active creep (the result of a continuously applied stress) on the MEEG fault system existed before the 1986 EPRI source model studies. For example, there is evidence that 37,000-year-old Pleistocene gravels are displaced above older Eocene deposits. In addition, at least seven earthquakes ranging in magnitude from a moment magnitude (M) of less than 3.0 to M 4.7 (based on historic felt reports and instrumental seismicity) are spatially coincident with the MEEG. Finally, the applicant stated that geodetic data measured over a 30-year period (before 1960) indicate an average normal slip rate of 0.43 centimeters (cm) (0.17 inches) per year.

The applicant's review of literature published since the 1986, EPRI studies found no new information indicating that the MEEG is a capable tectonic structure. In addition, the applicant stated that based on seismic reflection data, the MEEG terminates 4.8 to 6.4 km (3 to 4 mi) beneath the surface against Jurassic-age Louann salt deposits. The applicant concluded that based on the following facts, the MEEG is not a capable tectonic structure: (1) the MEEG does not penetrate the crystalline basement rock and therefore cannot be a source of moderate to large earthquakes; (2) seismic reflection data suggest that Quaternary deformation on the

MEEG is due to the movement of salt at depth; and (3) average slip rates of 0.43 cm (0.17 inches) per year do not represent slip rates associated with stable continental regions but can be explained by salt movement at depth. Finally, the applicant assumes that because most of the published data regarding the MEEG were available before 1986, the six EPRI teams had evaluated the data and concluded that the MEEG was not a capable tectonic feature.

*Balcones Fault Zone*

FSAR Subsection 2.5S.1.1.4.4.4.3, discusses the Balcones Fault Zone and states that major displacements on this feature took place during the middle Tertiary Period. The applicant stated that no data have been published since the 1986, EPRI study that clearly documents Quaternary deformation on the Balcones Fault Zone. However, one group of researchers (Collins et al., 1990) reported that weathered, most likely Pleistocene (1.8 Ma to the present), sedimentary fractures associated with individual faults within the zone may indicate Quaternary deformation on the Balcones faults. The applicant stated that the potential features discussed by Collins et al. do not provide a sufficient basis to categorize this fault zone as a capable tectonic structure. In addition, the applicant stated that Quaternary deformation on the Balcones Fault Zone is unlikely based on reports (also by Collins et al., 1990) that undeformed Quaternary terrace deposits overlie portions of this fault zone. The applicant concluded that there is no new information regarding the Balcones Fault Zone that necessitates a revision to the EPRI source zones.

*New Madrid Seismic Zone (NMSZ)*

The NMSZ is located more than 800 km (500 mi) from the STP site. This fault system extends from southeast Missouri to southwest Tennessee, is defined by three main fault segments, and covers an area approximately 220 km (125 mi) long and 40 km (25 mi) wide. The NMSZ produced at least three large earthquakes between December 1811, and February 1812. Magnitude estimates from these events range between M 7 and M 8. However, because of the considerable distance between the NMSZ and the STP site, the NMSZ only contributes to 1 percent of the hazard at the site (based on the 1986, EPRI study) (EPRI, 1986). Since the EPRI study, maximum magnitude ($M_{MAX}$) estimates for the NMSZ have remained consistent. However, the recurrence interval for large magnitude earthquakes in the NMSZ based on paleoseismic data was reduced from the 1,000 years used by the EPRI teams to the now widely accepted recurrence period of 500 years. The applicant's evaluation of the NMSZ, which is described in FSAR Section 2.5S.2, included this reduction in recurrence interval from 1,000 to 500 years.

*Quaternary Growth Faults*

The applicant stated that although evidence exists to support Quaternary deformation on growth faults in the STP site region, no new information has been published since the 1986, EPRI source model studies (EPRI, 1986) that would necessitate an update to the source models. In addition, these growth faults are understood to be confined to the overlying coastal plain section and not to penetrate the crystalline basement rock (Wheeler, 2005). Therefore, the applicant implies that these faults do not have the ability to generate significant earthquakes. The applicant concluded that gulf coastal growth faults are adequately accounted for in the EPRI seismic source models and no updates are required.

2.5S.1.2.2 Site Geology

FSAR Subsection 2.5S.1.2, summarizes the local site area: (1) physiography and geomorphology, (2) geologic history, (3) stratigraphy, and (4) structural geology. In addition, this section evaluates the site engineering geology, including the effects of human activities on the site area. As previously stated, the site area is defined for purposes of the geologic site characterization because the area is within an 8-km (5-mile) radius of STP, Units 3 and 4.

**Site Area Physiography and Geomorphology**

The STP site lies within the Coastal Prairies subprovince of the Gulf Coastal Plains Physiographic Province (previously described in FSAR Subsection 2.5S.1.1.1.1). Sands and clays of the Pleistocene-age Beaumont group extend across the entire site area and make up a majority of the surficial sediments. However, Holocene-age (10,000 years to the present) alluvial sediments overlie the Beaumont strata in a small portion of the eastern site area adjacent to the Colorado River. The applicant stated that the topographic relief across the site is generally less than 4.6 m (15 ft).

**Site Area Geologic History**

The applicant described the site area geologic history during the ongoing Quaternary Period (1.8 Ma to the present) as dominated by almost continuous sedimentary deposition that led to a gulfward migration of the shoreline. During the Pleistocene (1.8 Ma to 10,000 years ago), several glaciations took place that were each followed by interglacial episodes. The Beaumont Formation was deposited during an interglacial of the late Pleistocene Epoch, when sea levels were high and there was abundant alluvial and deltaic sedimentary deposition. FSAR Subsection 2.5S.1.1.2, contains a regional geologic description and additional details of these geologic events.

**Site Area Stratigraphy**

The applicant stated that approximately 12 km (40,000 ft) of sediment are present beneath the STP site area, nearly 8 km (26,000 ft) of which are Cenozoic coastal plain sediments. Approximately 4.2 km (14,000 ft) of older Mesozoic sediments overlie what is believed to be continental crust that forms the crystalline basement rock. In FSAR Subsection 2.5S.1.2.3, the applicant described the stratigraphic units underlying the STP site. As previously stated the Pleistocene Beaumont Formation underlies the STP site area and, in a few places, is covered by Holocene alluvial deposits. The applicant noted that the estimated thickness of the Beaumont Formation is approximately 122 m (400 ft) beneath the site. The exact thickness is unknown because the Beaumont is so similar in composition to the underlying Lissie Formation, which is also of Pleistocene age and a similar depositional environment.

The applicant performed 119 geotechnical borings and more than 30 cone penetrometer tests (CPTs) at the STP site, as part of its subsurface geologic investigations. FSAR Section 2.5S.4 provides a detailed description of the subsurface investigations at the site. Based on these investigations and previous investigations for STP, Units 1 and 2, the applicant divided the Beaumont formation into 12 strata based on the material properties, including soil designation and composition. In FSAR Subsection 2.5.S.1.2.3, the applicant describes the composition and hydrogeologic aspects of each soil stratum. FSAR Section 2.4S.12, includes a more detailed hydrogeologic description of the strata.

**Site Area Geologic Structures**

FSAR Subsection 2.5S.1.2.4, describes geologic structures within the site area, including basement structures and growth faults. The applicant stated that the continental crust that makes up the basement rock is interpreted to be "thin transitional crust" (i.e., a portion of the crust that has been exposed to considerable extension but not necessarily exposed to actual rifting). With regard to "discrete" faults or structures within the basement rock, the applicant concluded that no new information has been published about these structures since the 1986 EPRI studies. Buried growth faults associated with the Frio Fault Zone are the only geologic structures that exist within the STP site area. The applicant concluded that no growth faults project through the site location (defined as the 1-km [0.6-mile] site radius) or through the STP, Units 3 and 4, "footprint."

The following text summarizes the growth fault investigations that the applicant describes in FSAR Subsection 2.5S.1.2.4.2.

<u>*Growth Faults in the Site Area*</u>

The applicant describes growth fault investigations for STP, Units 1 and 2, as well as more recent investigations conducted for the STP, Units 3 and 4, COL application.

*Previous Growth Fault Studies in the Site Area*

The initial investigations for STP, Units 1 and 2, included: (1) aerial and high-altitude image interpretations, (2) analyses of boring data and geophysical well logs, (3) reviews of oil industry seismic reflection data, (4) analyses of lineaments, and (5) field investigations. The earlier investigations described ten growth faults in the site area, with seven of these faults interpreted to be buried beneath 1.5 km (5,000 ft) of undeformed sediment. The applicant determined that the other three faults may have been active during or since the Miocene Epoch (23.8 to 5.3 Ma). Based on seismic reflection data, two of these faults, growth faults "A" and "I," may approach within 300 meters (1,000 ft) of the ground surface. However, the seismic reflection profiles cannot further define the location of these faults above 150 meters (500 ft) due to the limits of resolution for the data. Based on subsurface data, remote sensing imagery, and undeformed strata exposed in an excavated channel where growth fault "I" is inferred to project, the UFSAR for STP, Units 1 and 2, found no evidence that growth fault "A" or "I" projects to the surface.

*Updated Information on Growth Faults in the Site Area*

The applicant compiled data from seven sources as part of the growth fault investigation for STP, Units 3 and 4, including the UFSAR for STP, Units 1 and 2. FSAR Table 2.5S.1-1, "Growth Faults within the Greater Site Vicinity," lists all of the faults documented in the seven sources, and FSAR Subsection 2.5S.1.2.4.2.2.1, describes the findings of these studies. The applicant stated that most of the faults described in the studies and depicted in SER Figure 2.5S.1-4 (based on FSAR Figure 2.5S.1-42, "Site Vicinity (25-mile radius) Growth Faults and Growth Fault Surface Projections,") can be projected to the surface. FSAR Figure 2.5S.1-43, "Site Vicinity (5-mile radius) Growth Fault Surface Projections," shows those growth faults (from three of the seven investigations) that are inferred to project to the surface within the 8-km (5-mi) site radius. The applicant noted that there is uncertainty, on the order of several miles, associated with projecting growth faults at depth to the surface. In addition, based on the applicant's descriptions of the seven existing growth fault investigations, the resolution limits associated with some of the data do not allow the growth faults to be identified in the shallow surface or even at depths of less than approximately 1.8 km (6,000 ft) beneath the surface.

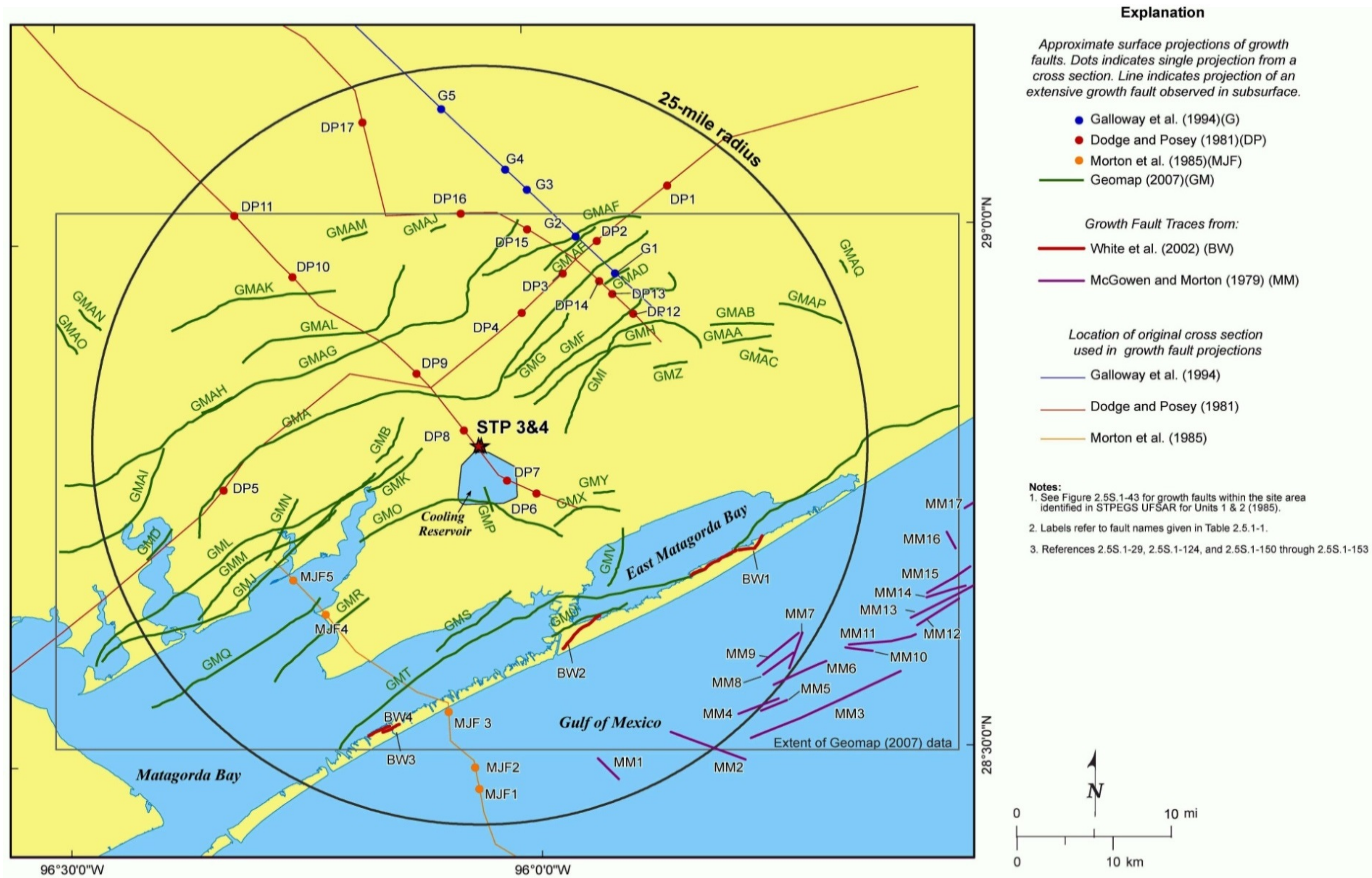


**Figure 2.5S.1-4 Map of Growth Faults and Growth Fault Surface Projections within the STP Site Vicinity (FSAR Figure 2.5S.1-42)**

In addition to the compilation of existing data, the applicant performed new aerial photographic analyses as well as aerial and field reconnaissance investigations (including lineament analyses) to evaluate growth faults in the STP site area. The applicant focused the investigations on growth faults “A” (Matagorda STP 12A) and “I” (Matagorda STP 12I), identified in the UFSAR for STP 1 and 2, because their inferred surface projections lie within the STP site area and there is evidence that they may deform strata younger than 5.3 Ma. The applicant examined linear features in the STP site area and investigated spatial associations between these features and the inferred surface projections of known growth faults. In conclusion, the applicant found that distinct linear features are associated with the surface projections for Growth fault Matagorda STP 12I, but such features are less pronounced or nonexistent for other growth faults within the site area. The applicant conducted an aerial reconnaissance to investigate linear or topographic features from above but found no evidence for such features within the 8-km (5-mi) site radius.

Geomap Company published structural contour maps (Geomap, 2007) that showed the deformation of Miocene (23.8 to 5.3 Ma) strata interpreted as a result of growth faulting at depth. The surface projection of one of these growth faults, Matagorda GMO, is coincident with the surface projection of Matagorda STP 12I. Based on exposed topographic breaks and spatial coincidence, the applicant concluded that these two faults (Matagorda GMO and Matagorda STP 12I) most likely represent the same growth fault. The applicant also notes that Geomap fault GMA is likely coincident with STP 12A.

Because growth fault GMO/Matagorda STP12I projects beneath the southwest corner of the STP main cooling reservoir and within the site area, the applicant conducted four surveys across the fault to look for evidence of fault rupture or continuous deformation along the fault. SER Figure 2.5S.1-5 (reproduced from FSAR Figure 2.5S.1-45, “Growth Fault Projections and Lineaments”), shows the locations of the four surveys in the southern portion of the site area west of the main cooling reservoir. The applicant discussed the topographic profiles associated with each of the four surveys as depicted in FSAR Figure 2.5S.1-46, “Field Survey Elevation Profiles.” The applicant identified no discrete fault rupture surfaces along any of the profiles. However, the applicant did see evidence for broad surface flexure across at least three of the four survey profiles. Survey STP L4 is the closest to the main cooling reservoir and the applicant saw no “clear” topographic changes across this profile, especially in comparison to the other three surveys. Therefore, the applicant was not able to confirm that the projection of growth fault GMO/Matagorda STP 12I extends into the STP cooling water reservoir.

The applicant stated that one other growth fault, GMP (identified by the Geomap Company), projects close to the STP cooling reservoir. This growth fault trends north to northeast and projects through the southern portion of the main cooling reservoir. This growth fault is identified in the investigation for STP, Units 3 and 4, but it was not described in previous growth fault studies. The applicant concluded that there is no surficial evidence associated with this fault to suggest recent deformation.

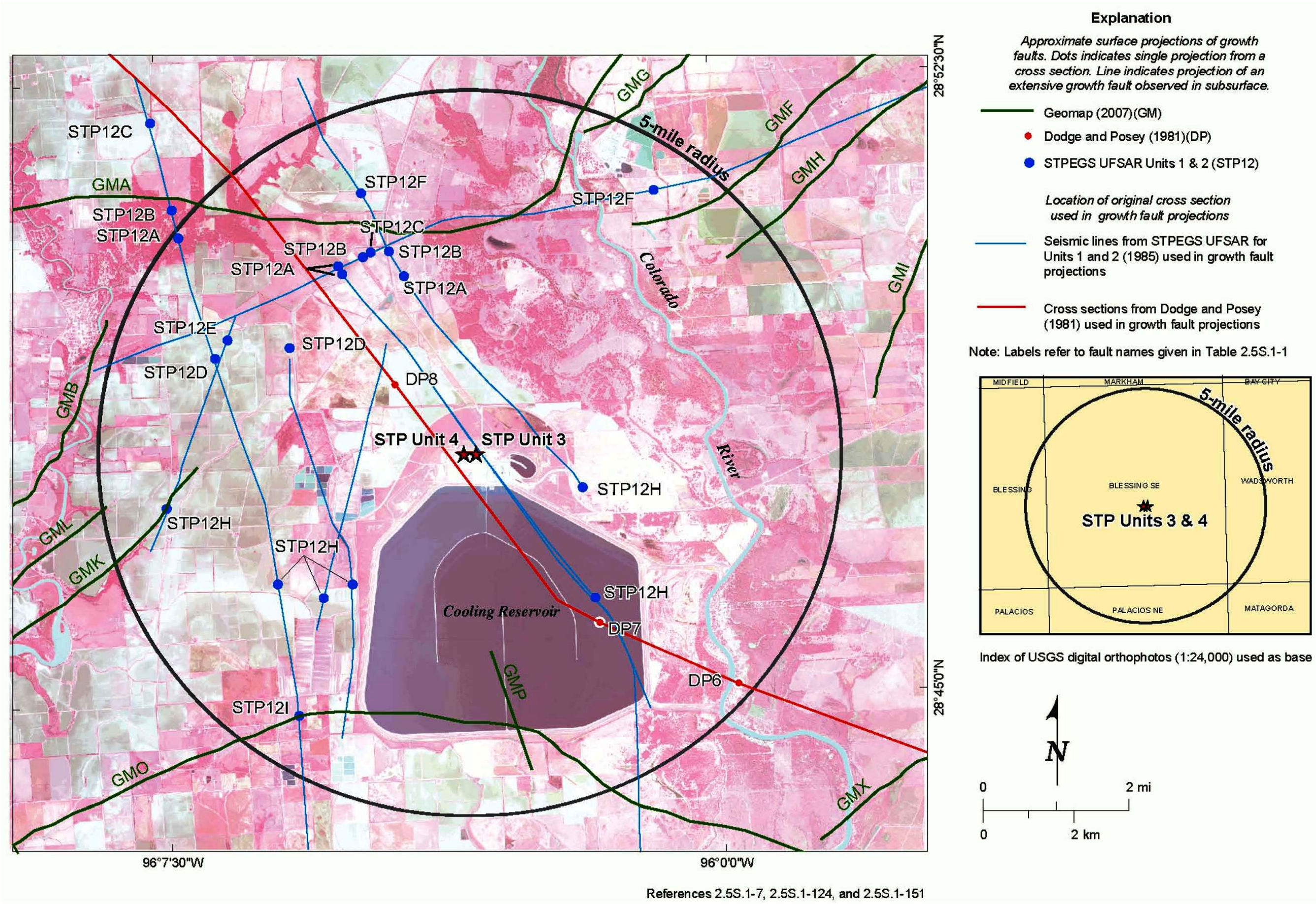


**Figure 2.5S.1-5 Map of Growth Fault Projections, Lineaments, and Topographic Survey Points Within the STP Site Area (Reproduced from FSAR Figure 2.5S.1-45)**

**Site Area Geologic Hazard Evaluation**

FSAR Subsection 2.5S.1.2.5, discusses potential geologic hazards at the STP, Units 3 and 4, site. The applicant concluded that there is no evidence for dissolution, zones of deformation, or volcanic activity within the STP site area.

**Site Engineering Geology Evaluation**

FSAR Subsection 2.5S.1.2.6, discusses the applicant's evaluation of the site's engineering geology, including potential effects of human activities at the STP site. The applicant discussed the engineering soil properties and the behavior of foundation materials in FSAR Section 2.5S.4. The applicant concluded that there is no evidence for weathering or dissolution at the STP site. Furthermore, there is also no evidence of deformational zones, capable tectonic structures; or previous earthquake activity at the STP site. The applicant stated that it will conduct excavation mapping during construction to evaluate any potential features beneath the site.

FSAR Subsection 2.5S.1.2.6.5, discusses the effects of human activities at the STP site, specifically the effects of oil and ground water withdrawal that could lead to subsidence of the underlying sedimentary units. After calculating the anticipated maximum subsidence at the STP site due to construction dewatering, the applicant concluded that the calculated values of 0.04 to 0.05 ft are not likely because some of the extracted water will be replaced by storm water or runoff. The applicant discussed construction dewatering in FSAR Section 2.5S.4, as it relates to geotechnical engineering. The applicant stated that no mining or "excessive" ground water injection takes place in the STP site area. FSAR Section 2.4S.12, "Groundwater," discusses ground water conditions and the effects of human activities related to ground water in more detail.

### *2.5S.1.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the basic geologic and seismic information, and the associated acceptance criteria, are in Section 2.5.1 of NUREG–0800. The acceptance criteria for reviewing COL License Information Item 2.23, are in Section 2.5.1 of NUREG–0800.

In particular, the applicable regulatory requirements for reviewing geologic and seismic information are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying geologic site characteristics with appropriate consideration of the most severe of the natural phenomena historically reported for the site and surrounding area and with a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data were accumulated.
- 10 CFR Part 100, Section 100.23, "Geologic and seismic siting criteria," for evaluating the suitability of a proposed site based on considerations of geologic, geotechnical, geophysical, and seismic characteristics of the proposed site. Geologic and seismic siting factors must include the SSE for the site and the potential for surface tectonic and non-tectonic deformation. The site-specific GMRS must satisfy the requirements of 10 CFR 100.23, with respect to the development of the SSE.

The related regulatory requirements and acceptance criteria in Section 2.5.1 of NUREG–0800 are as follows:

- Regional Geology: In meeting the requirements of 10 CFR 52.79 and 10 CFR 100.23, FSAR Subsection 2.5S.1.1, will be considered acceptable if a complete and documented discussion is presented for all geologic (including tectonic and nontectonic), geotechnical, seismic, and geophysical characteristics, as well as conditions caused by human activities deemed important for the safe siting and design of the plant.

- Site Geology: In meeting the requirements of 10 CFR 52.79 and 10 CFR 100.23 and the regulatory positions in RGs 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion"; 1.132, "Site Investigations for Foundations of Nuclear Power Plants"; 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants"; 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites"; 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)"; and 4.7, "General Site Suitability Criteria for Nuclear Power Stations"; FSAR Subsection 2.5S.1.2, "Site Area Geology"; will be considered acceptable if it contains a description and evaluation of geologic (including tectonic and non-tectonic) features; geotechnical characteristics; seismic conditions; and conditions caused by human activities at appropriate levels of detail within areas defined by circles drawn around the site using radii of 40 km (25 mi) for site vicinity, 8 km (5 mi) for site area, and 1 km (0.6 mi) for site location.

### *2.5S.1.4 Technical Evaluation*

The staff reviewed the information in Section 2.5S.1 of the STP, Units 3 and 4, COL FSAR:

*COL License Information Items*

- COL License Information Item 2.23 Geology, Seismology, and Geotechnical Engineering

The staff reviewed the information provided by the applicant in FSAR Section 2.5S.1, to address COL License Information Item 2.23. The specific information includes the description and evaluation of regional and site geologic and seismic data collected by the applicant during site and regional investigations.

This SER section presents the staff's evaluation of the geologic and seismic information submitted by the applicant in FSAR Section 2.5S.1. The technical information in FSAR Section 2.5S.1, are results of the applicant's surface and subsurface geologic, seismic, and geotechnical investigations, which were undertaken at increasing levels of detail closer to the site. The staff's review determined whether the applicant has complied with the applicable regulations and whether the applicant has conducted these investigations with the appropriate levels of detail within the four circumscribed areas designated in RG 1.208, which are defined based on various distances from the site (i.e., circular areas drawn with radii of 320 km [200 mi], 40 km [25 mi], 8 km [5 m], and 1 km [0.6 mi] from the site).

FSAR Section 2.5S.1, contains geologic and seismic information collected by the applicant in support of the vibratory ground motion analysis and the site-specific GMRS in FSAR Section 2.5S.2. RG 1.208, recommends that applicants update the geologic, seismic, and geophysical database and evaluate any new data to determine whether revisions to the existing seismic source models are necessary. Consequently, the staff's review focused on geologic and seismic data published since the late 1980s, to assess whether these data indicate a need for changes to the existing seismic source models.

To thoroughly evaluate the applicant's geologic and seismic information, the staff obtained the assistance of the USGS. The staff and its USGS counterparts visited the STP site in August 2008, to confirm the applicant's interpretations, assumptions, and conclusions related to potential geologic and seismic hazards. The staff's evaluation of the applicant's information in COL FSAR Section 2.5S.1, and the applicant's responses to the staff's RAIs are presented below.

2.5S.1.4.1 Regional Geology

The staff's review focused on STP, Units 3 and 4, COL FSAR Subsection 2.5S.1.1, the applicant's description of the regional physiography, geomorphology, geologic history, stratigraphy, and tectonic setting within a 320-km (200-mi) radius of the STP site. The following SER subsections present the staff's evaluation of the applicant's information in FSAR Subsection 2.5S.1.1, and in the applicant's responses to RAIs.

**Regional Physiography and Geomorphology**

FSAR Subsection 2.5S.1.1.1, discusses the regional physiography and geomorphology surrounding the STP site. The applicant stated that the site is located within the Coastal Prairie subsection of the Gulf Coastal Plain Physiographic Province. The Coastal Prairie subprovince, where the STP site is located, makes up the southeastern portion of the Gulf Coastal Plain. The applicant noted that growth faulting is common in the gulf coastal region where sedimentary units have experienced rapid deposition. The staff reviewed FSAR Subsection 2.5S.1.1.1, and concluded that the applicant has provided a thorough and accurate description of the physiographic and geomorphic features in the site region to support the STP COL application and in accordance with RG 1.208.

**Regional Geologic History**

The staff reviewed FSAR Subsection 2.5S.1.1.2.6, the applicant's description of the Cenozoic (65 Ma to the present) geologic history. The applicant explains that the loading of the crust due to a rapid seaward deposition of sediments during the Cenozoic Era led to the subsidence of the Gulf of Mexico Basin that has continued through to the present. As a result of rapid sedimentation during Cenozoic times, growth faults developed throughout the coastal region.

The applicant provided more detailed discussions of growth faulting in FSAR Subsection 2.5S.1.1.4, and FSAR Subsection 2.5S.1.2.4. The staff reviewed FSAR Subsection 2.5S.1.1.2, and concluded that the applicant has provided a thorough and accurate description of the regional geologic history, including key depositional processes in the STP site region in support of the STP, Units 3 and 4, COL application and in accordance with RG 1.208.

**Regional Stratigraphy**

The staff's review focused on FSAR Subsection 2.5S.1.1.3, the applicant's description of the Quaternary stratigraphic units in the site region. FSAR Subsection 2.5S.1.1.3.5.6, describes the Beaumont Formation, the upper Pleistocene (1.8 MA to 10 ka) stratigraphic unit that is composed of alluvial fan deposits. This unit underlies the STP site and is approximately 122 meters (400 ft) thick, according to FSAR Figure 2.5S.1-13, "General Cenozoic Stratigraphic Column." The applicant stated that the actual thickness of the Beaumont is difficult to confine because the thickness is variable, and the composition is similar to that of the underlying lower Pleistocene Lissie Formation.

FSAR Figure 2.5S.1-14, "Fluvial Deposits of the Colorado River," illustrates the Colorado River Fluvial deposits within the STP site vicinity and identifies the following three units of the Beaumont Formation: (1) Bay City, (2) El Campo, and (3) Lolita Valley fills. However, FSAR Section 2.5S.1, does not discuss these valley fills adjacent to the STP site, so the staff issued RAI 02.05.01-16, requesting the applicant to explain the three units and their significance to the site's geology. In its response to RAI 02.05.01-16, dated July 9, 2008 (ML081960070), the applicant thoroughly described the valley fills based on numerous recent publications (including Blum and Aslan [2006] and Aslan and Blum [1999]). According to the applicant's response, the fill deposits are significant because they define depositional sequences of the Beaumont Formation that took place from about 320,000 to just over 100,000 years ago. The STP site lies within the Bay City Valley fill, which was likely deposited between 100,000 and 150,000 years ago. Based on the applicant's response to RAI 02.05.01-16, the staff concluded that the applicant has provided a thorough description of the valley fill deposits associated with the Beaumont Formation. Therefore, RAI 02.05.01-16 is resolved and closed.

After reviewing STP COL FSAR Subsection 2.5S.1.1.3, and the response to RAI 02.05.01-16, the staff concluded that the applicant has provided an adequate description of the regional stratigraphy in support of the STP COL application.

**Regional Tectonic Setting**

STP COL FSAR Subsection 2.5S.1.1.4, describes the regional tectonic setting within a 320-km (200-mi) radius of the STP site. Within this FSAR section, the applicant discusses: (1) the regional tectonic history, (2) the regional tectonic stress environment, (3) regional gravity and magnetic features, and (4) regional tectonic structures. The applicant concluded in FSAR Subsection 2.5S.1.1.4, that there is no new evidence for Quaternary seismic activity on any known geologic structure, and there is no new available information that compels a significant revision to the 1986, EPRI seismic source models for the site region.

The staff issued three RAIs requesting clarifications of and editorial corrections in figures and text in FSAR Subsection 2.5S.1.1.4. In RAI 02.05.01-2, the staff asked the applicant to correctly label gravity features in FSAR Figures 2.5S.1-15, "Gravity Anomaly Features in Site Region (200-mile radius)," and 2.5S.1-20, "Regional Gravity Anomaly Map." In RAI 02.05.01-5, the staff asked the applicant to revise incorrect FSAR section cross references throughout FSAR Subsection 2.5S.1.1.4. In RAI 02.05.01-17, the staff asked the applicant to clarify symbols and colors used to identify earthquakes and salt diapers in FSAR Figure 2.5S.1-17, "Tectonic Features in Site Region (200-mile radius)." In its response to RAI 02.05.01-5, dated June 26, 2008 (ML081970231) and RAIs 02.05.01-2 and 02.05.01-17, dated July 9, 2008, the applicant provided all corrections and clarifications. Therefore, RAI 02.05.01-2, RAI 02.05.01-5, and RAI 02.05.01-17, are resolved and closed.

Regional Tectonic History

FSAR Subsection 2.5S.1.1.4.1, states that the STP site lies in the Gulf Coastal Plain physiographic province that formed during the opening of the Gulf of Mexico Basin during the Mesozoic Era (248 to 65 Ma). The crustal material beneath the STP site, buried by approximately 12 km (40,000 ft) of Cenozoic and Mesozoic sediments, was mostly influenced by collisional tectonic events during the Paleozoic Era (354 to 248 Ma). An inland uplift of the Cordillera (north and west of the STP site) during the Quaternary Period led to massive sedimentary deposition and subsidence toward the Gulf of Mexico Basin.

The applicant briefly discusses deposition of the late-Pleistocene sedimentary Beaumont Formation in response to glacial melting during the Sangamon Interglacial. The staff noted that a number of papers published during the past 15 years, including Blum and Aslan (2006), discuss the potential tilting of Pleistocene surfaces in the region surrounding the STP site. Because tilting can be an indicator of fault movement, the staff issued RAI 02.05.01-1, requesting the applicant to provide an up-to-date summary of the Quaternary sediments and their relation to the tectonic history of the site region.

In its response to RAI 02.05.01-1, dated July 24, 2008 (ML082100162), the applicant provided a more thorough description of Pleistocene and Holocene sediments of the Quaternary Period based on the recent literature. The applicant discussed mapped Pleistocene surfaces that demonstrate tilting at increasingly higher angles coincident with the increasing age of the surfaces. In other words, the oldest Pleistocene surfaces are farther inland from the younger surfaces and have steeper slopes. The RAI response attributes this increased tilting to high rates of sedimentary deposition toward the Gulf that led to a flexural response and uplift of the older inland surfaces. The applicant stated that the increase in sedimentary deposition began in the Late Jurassic Period following a period of extension and rifting in the Gulf of Mexico, which has continued into the geologic present.

The staff reviewed the applicant's response to RAI 02.05.01-1, and verified that the information is consistent with the most current understanding of Quaternary stratigraphy for the STP site region. The staff concluded that the applicant has provided a more comprehensive description of the youngest sediments present in the site region and has adequately discussed these sedimentary units with respect to the regional structural geology. Therefore, RAI 02.05.01-1 is resolved and closed.

The staff reviewed FSAR Subsection 2.5S.1.1.4.1, and the applicant's response to RAI 02.05.01-1. The staff concluded that the applicant's characterization of the tectonic history for the STP site region adequately supports the COL application.

Regional Tectonic Stress

In STP COL FSAR Subsection 2.5S.1.1.4.2, the applicant stated that the tectonic stress in the CEUS, including the gulf coastal region, is primarily characterized by a compressive stress field with a principal horizontal shear direction oriented northeast and southwest. The applicant stated that this characterization of the regional tectonic stress is consistent with the most updated World Stress Map (Reinecker et al., 2005). Localized stresses (such as flexural loading and buoyancy forces) may also be influencing the STP site region. The staff reviewed

FSAR Subsection 2.5S.1.1.4.2, and concluded that the applicant's characterization of the regional tectonic stresses influencing the STP site adequately supports the COL application.

Regional Gravity and Magnetic Data

The staff's review of FSAR Subsection 2.5S.1.1.4.3, focused on the applicant's description of features identified in the gravity and magnetic data analyzed for the COL application. The applicant stated that gravity and magnetic anomalies identified in the available data correspond with major tectonic events discussed in the FSAR. In addition, the applicant identifies ten individual gravity features and six individual magnetic features described in FSAR Subsections 2.5S.1.1.4.3.1 and 2.5S.1.1.4.3.2. According to FSAR Subsection 2.5S.1.1.4.3, there are no data that identify new or unknown geologic structures within the STP region. The staff reviewed the data in FSAR Subsection 2.5S.1.1.4.3, and concluded that the applicant adequately evaluates a range of currently available gravity and magnetic data in support of the STP, Units 3 and 4, COL application.

Principal Tectonic Structures

FSAR Subsection 2.5S.1.1.4.4, discusses principal regional tectonic structures in the STP site region based on information published since the 1986 EPRI study. The applicant concluded that none of this more recent information justifies a "significant change in the EPRI seismic source model."

In FSAR Subsection 2.5S.1.1.4.4.3, the applicant briefly discusses Jurassic (206 to 144 Ma) faulting of basement rock due to extension and rifting. The application states that "basement block bounding faults" formed during the Jurassic rifting and extension "have been interpreted within both the thick and thin transitional crust." The STP site is located above thin transitional crust while the northwest portion of the site region is underlain by thick transitional crust. The staff noted that the geologic setting and tectonic history of much of the site region are similar to other regions where large historic earthquakes have occurred, such as Charleston, South Carolina. Therefore, the staff issued RAI 02.05.01-3, asking the applicant to provide additional information on the strong earthquake potential for thick and thin transitional crustal structures beneath the STP site region.

In its response to RAI 02.05.01-3, dated August 27, 2008 (ML082490086), the applicant explained that the seismic hazard in the STP site region is modeled using areal source zones determined by the EPRI-seismicity owners group (SOG) earth science teams (EST) in the mid-1980s. The ESTs used areal source zones due to a lack of evidence for "discrete faults that may be potential seismic sources" in the STP site region. Regarding the potential for the site to experience large magnitude earthquakes similar to those experienced in other parts of the CEUS, the applicant stated the following:

> An explicit motivation for the EPRI-SOG study as stated within the preface to the source characterizations reports (Reference 1) was to assess the possibility for an earthquake similar to that which occurred near Charleston throughout the CEUS.

The applicant also reviewed: (1) recent gravity and magnetic data, (2) recent kinematic models for the Gulf of Mexico, (3) revised stress models, and (4) seismicity data since the mid-1980s. The applicant elaborated on these investigations in FSAR Sections 2.5S.1 and 2.5S.2. The applicant described this effort as a "comprehensive review of all available information and data" since the EPRI study in the 1980s. This review intentionally looked for relevant studies dealing with “thick- and thin-transitional crust beneath the site region.” Based on these investigations, the applicant stated that the new information necessitated modifications to the maximum magnitudes for some of the gulf coastal seismic sources identified in the EPRI-SOG study. The applicant identified additional updates to the EPRI-SOG model that are discussed in FSAR Section 2.5S.2. The staff’s evaluation is in Section 2.5S.2, of this SER.

The staff reviewed the applicant’s response to RAI 02.05.01-3, the gravity and magnetic data for the site region, and relevant publications. The staff concluded that the applicant has followed NRC guidance set forth in RG 1.208 and has appropriately used the EPRI-SOG source models as a starting point for evaluating seismic source zones in the STP site region. The staff further concluded that the applicant has adequately incorporated more recent geologic information such as stress data, kinematic data, and gravity and magnetic data into the evaluation of the transitional crust located beneath the STP site. Finally, the staff concluded that although there is no direct evidence for faulting within the STP site area, the applicant has accounted for earthquakes greater than M 5 in the PSHA for the STP site. The staff’s evaluation of the applicant’s PSHA is in Section 2.5S.2 of this SER. Therefore, RAI 02.05.01-3 is resolved and closed.

<u>Tertiary Tectonic Structures</u>

FSAR Subsection 2.5S.1.1.4.4.4, discusses early Cenozoic (Tertiary) salt structures, growth faults, and “basement-involved” faults. The applicant indicates that sedimentary processes dominated during the Tertiary Period and that major tectonic events did not impact the STP site region during that time.

*Tertiary Salt Structures*

FSAR Subsection 2.5S.1.1.4.4.4.1, describes Tertiary salt structures within the STP site region. The applicant stated that three salt domes (Big Hill, Hawkinsville, and Markham) are identified in the STP site vicinity and that the closest of these salt domes is approximately 16 km (10 mi) from proposed STP, Units 3 and 4. The applicant concluded that no salt structures are identified within the immediate STP site. The staff concluded that the applicant has adequately evaluated salt structures within the STP site region and has presented no evidence that suggests the potential for salt deformation beneath the proposed STP, Units 3 and 4.

*Tertiary Growth Faults*

FSAR Subsection 2.5S.1.1.4.4.4.2, describes five Tertiary growth fault zones within the STP site region that trend east-northeast to south-southwest and are coincident with the trend of the Gulf of Mexico shoreline. The applicant indicates that as the shoreline migrated toward the Gulf since the late Mesozoic Era, the active growth fault zones also migrated. The applicant stated that none of the growth fault zones within the site region penetrates into the crystalline basement rock. Instead, they all merge into detachment horizons (weak stratigraphic layers) of salt and/or shale. The late Oligocene (approximately 23.8 Ma) Frio Fault Zone is located closest to the STP site. The applicant noted that the Frio is approximately 60 km (37 mi) wide and terminates against a deep detachment horizon. FSAR Subsection 2.5S.1.2.4.2, includes a more detailed description of growth faults near the STP site, and Subsection 2.5S.1.4.2, of this SER includes the staff’s evaluation of these growth faults.

*Tertiary Basement Involved Faults*

In FSAR Subsection 2.5S.1.1.4.4.4.3, the applicant described the Luling and Balcones Fault Zones that form a northeast-southwest trending graben system subparallel with the buried Ouachita Orogenic Belt. The applicant suggested that these faults may be controlled by pre-existing faults at depth that originally formed during the Paleozoic Ouachita Orogeny, or they may have formed during the Miocene Epoch due to tensile stresses. The applicant does not favor one explanation over the other. Neither of these faults shows convincing evidence for displacement since the Tertiary Period. However, one group of authors (Collins et al., 1990) speculates that there may be Quaternary deformation features associated with the Balcones Fault Zone. Therefore, in FSAR Subsection 2.5S.1.1.4.4.5.2, the applicant provides an additional description of the Balcones with regard to the Quaternary deformation.

The staff reviewed FSAR Subsection 2.5S.1.1.4.4.4, and concluded that the applicant has adequately described Tertiary tectonic and non-tectonic structures in the STP site region and has evaluated these structures for evidence of post-Quaternary activity. The staff concluded that the applicant has presented no definitive evidence for post-Quaternary activity on any of the described Tertiary structures. The staff's evaluation of potential Quaternary deformation associated with the Balcones fault is presented later in this SER Section.

<u>Quaternary Tectonic Structures</u>

In FSAR Subsection 2.5S.1.1.4.4.5, the applicant concluded that there is no new information regarding Quaternary activity associated with any tectonic features in the site region that requires a revision or update of the EPRI seismic source characterization for the gulf coastal region.

*Mt. Enterprise-Elkhart Graben (MEEG) System*

In FSAR Subsection 2.5S.1.1.4.4.5.1, the applicant discusses normal faults of the MEEG system that are located just within the northern perimeter of the site region. The applicant concluded that the MEEG system is not a capable tectonic source based on the fact that: (1) the MEEG likely does not penetrate the crystalline basement rock and therefore, it is not a source of moderate to large earthquakes; (2) seismic reflection data suggest that Quaternary deformation on the MEEG is due to the movement of salt at depth; and (3) average slip rates of 0.43 cm (0.17 inches) per year do not reflect typical slip rates associated with stable continental regions, but they can be explained by salt movement at depth. Finally, the applicant assumed that because most of the published data regarding the MEEG were available before 1986, the six EPRI teams had evaluated the data and concluded that the MEEG is not a capable tectonic feature.

FSAR Subsection 2.5S.1.1.4.4.5.1, explains that normal faults of the MEEG displace gravel of late Quaternary age. The discussion also says that seismic reflection data indicate that the faults are rooted in Jurassic salt, and the movement of salt at depth probably drives slip on the faults. Geodetic leveling data suggest an average slip (displacement) rate across the MEEG of approximately 4 millimeters per year (mm/yr) (0.17 inches/yr) measured over a 30-year period. The staff issued RAI 2.05.01-4, requesting the applicant to provide a more detailed summary of the data evaluating Late Quaternary faulting on the MEEG and to further explain the basis for the assumption that salt movement is driving deformation within the MEEG system. The staff also asked the applicant in RAI 02.05.01-4, to explain whether or not salt movement at depth could produce a modern slip of 4 mm/yr (0.17 inches/yr) on overlying normal faults, and whether stratigraphic relations of the displaced gravel favor sudden surface displacement or gradual creep.

In its response to RAI 02.05.01-4, dated September 4, 2008 (ML082530449), the applicant discussed evidence for Quaternary movement on the MEEG fault system, as reported by Collins et al. (1980). This research discussed folded Quaternary sand and gravel deposits (about 37,000 years old) that overlie faulted Eocene sands in the westernmost portion of the MEEG. The authors did not document faulting of the Quaternary deposits but noted that the Quaternary deposits were folded above the Eocene faults. The applicant stated that based on the evidence presented by these authors, including the apparent absence of a colluvial wedge that might indicate sudden movement on a fault, the slip was likely gradual rather than sudden, thus favoring salt movement at depth as the driving mechanism. With regard to the estimated 4mm/yr (0.17 inches/yr) displacement across the MEEG, the applicant stated that Quaternary displacement rates were estimated using offsets in the Quaternary sands and gravels, offsets observed in an auger profile, and offsets observed from the geodetic leveling data. The geodetic leveling data produced the largest estimates of offsets. The applicant does not know the accuracy or uncertainty of the leveling data. However, slip rates similar to 4 mm/yr (0.17 in/yr) are common in areas of Louisiana and the Gulf of Mexico where salt movement is known to deform overlying strata.

The staff reviewed the applicant's response to RAI 02.05.01-4, and concluded that the applicant has provided a more detailed summary of the geologic data to support late Quaternary deformation associated with the MEEG system. The staff concluded that the applicant has provided adequate justification to support the interpretation in the FSAR that deformation on the MEEG is due to active salt movement and is likely not an active tectonic structure. Finally, the staff concluded that the applicant has cited appropriate examples of similar rates of deformation (on the order of 2 to10 mm per year) due to salt movement at depth in other areas of the Gulf coast to support a separation rate of 4 mm/yr (0.17 in/yr) across the MEEG system. Therefore, RAI 02.05.01-4 is resolved and closed.

*Balcones Fault Zone*

FSAR Subsections 2.5S.1.1.4.4.4.3 and 2.5S.1.1.4.4.5.2, discuss the Balcones Fault Zone, a northeast-southwest trending fault system that lies approximately 225 km (140 mi) northwest of the STP site. The applicant stated that major displacements on this feature took place during the middle Tertiary (5.3 to 33.7 Ma). One group of researchers (Collins et al., 1990) reported that weathered (most likely Pleistocene, 1.8 Ma to the present) sedimentary fractures associated with individual faults within the zone may indicate Quaternary deformation on the Balcones faults and that a paleoseismic study is needed to determine whether the Balcones Fault Zone is active. However, the applicant concluded that the evidence for Quaternary deformation on the Balcones Fault Zone (as presented by Collins et al., 1990) is "equivocal" and does not constitute a revision to the EPRI seismic source models for the Gulf Coastal Plain.

Because a large magnitude earthquake within the Balcones Fault Zone could potentially cause surface deformation at distances that would include the STP site, the staff issued RAI 02.05.01-6, requesting the applicant, to justify why a paleoseismic investigation was not conducted to evaluate the potential for Quaternary deformation on the Balcones Fault Zone. In its response to RAI 02.05.01-6, dated July 16, 2008 (ML082030326), the applicant stated that it followed the guidance in RG 1.208 for developing its seismic source model for the STP site.

RG 1.208 states that seismic sources defined by the EPRI-SOG study in the mid-1980s (EPRI, 1986; 1989) may be used as a starting point for an applicant's seismic source characterization provided that the applicant evaluates any new information developed after the EPRI-SOG study. The applicant pointed out that the Collins et al. (1990) study was published after the EPRI-SOG study and was therefore evaluated for the STP, Units 3 and 4, COL application. The applicant also states that the Collins et al. (1990) document is the only post-EPRI report citing the association with the potential Quaternary deformation and the Balcones Fault Zone. The applicant also notes that the Collins et al. (1990) report is a field trip guidebook published by the Austin Geological Society and is not considered a peer-reviewed publication. Furthermore, the evidence reported in the guidebook is speculative and is not based on documented field evidence.

With regard to the "wedge shaped," sediment-filled fractures identified by Collins et al. (1990), the applicant stated that this evidence alone does not qualify the Balcones Fault Zone as a capable tectonic source, because such fractures "can be explained by non-tectonic processes." Furthermore, "poorly dated Pleistocene high terrace deposits" overlying the fault strands "are apparently not offset by the fault," thus making it unlikely that the Balcones faults have moved in the past hundreds of thousands of years. In conclusion, the applicant stated that the "equivocal" evidence provided by Collins et al. (1990) "does not reflect a change in the state of knowledge of the seismic potential of the Balcones Fault Zone that is robust enough to justify either modifying the seismic source characterizations of the EPRI-SOG model, or conducting a detailed paleoseismic study."

To further support the response to RAI 02.05.01-6, the applicant contacted and interviewed the lead author, Eddie Collins (Collins, 2008), of the referenced guidebook (Collins et al., 1990). Mr. Collins provided the following statement to the applicant regarding evidence for recent geologic activity on the Balcones Fault Zone: "I don't know of any field evidence that would verify a Pleistocene or Holocene slip on any of the fault strands that compose the Balcones Fault Zone."

Based on its evaluation of the Collins et al. (1990) report and personal communication with the lead author (Eddie Collins), the applicant determined that no additional evaluation of the Balcones Fault Zone (such as a paleoseismic investigation) is warranted, because "there is no new evidence to support the conclusion that the Balcones fault zone is a capable tectonic feature." The applicant stated that at least two of the six EPRI-SOG ESTs (Bechtel and Law Engineering) include the Balcones Fault Zone in their seismic source characterizations for the CEUS, and "thus the seismogenic potential of the Balcones fault zone as understood at the time of the EPRI-SOG study is reflected in the EPRI-SOG source model for the central and eastern US."

The staff reviewed the applicant's response to RAI 02.05.01-6, and concluded that although the evidence in Collins et al. (1990) does suggest that an additional investigation of the Balcones Fault Zone may be warranted, the evidence is questionable and does not necessitate the need for the applicant to perform a paleoseismic investigation for the STP site. Furthermore, EPRI-SOG ESTs were aware of the Balcones Fault Zone, and the age of the most recent deformation across the Balcones faults was questionable at that time. To date, there is no documented evidence for post-Tertiary movement (younger than 1.8 Ma) on the Balcones Fault Zone. As noted by the applicant's RAI response, at least two of the EPRI-SOG ESTs included the Balcones Fault Zone in seismic source characterizations. Subsection 2.5S.2.4 of this SER includes the staff's evaluation of the seismic source characterizations that the applicant incorporated into its PSHA for the STP site. Therefore, RAI 02.05.01-6 is resolved and closed.

*New Madrid Seismic Zone (NMSZ)*

The NMSZ is located more than 800 km (500 mi) from the STP site. Because of the considerable distance between the NMSZ and the STP site, the NMSZ only contributes to one percent of the hazard at the site based on the 1986, EPRI study. Since the EPRI study, $M_{MAX}$ estimates for the NMSZ have remained consistent. However, the recurrence interval for large magnitude earthquakes in the NMSZ—based on paleoseismic data—was reduced from the 1,000 years the EPRI teams used to the now widely accepted recurrence period of 500 years. The applicant included this reduction in the recurrence interval in the seismic evaluation of the NMSZ, which FSAR Section 2.5S.2 describes and the staff evaluates in SER Subsection 2.5S.2.4. Based on this review of FSAR Subsection 2.5S.1.1.4.4.5.3, the staff concluded that the applicant has provided an adequate description of the NMSZ with respect to the regional tectonic setting for the STP site and in accordance with RG 1.208. SER Section 2.5S.2 includes the staff's evaluation of the NMSZ with respect to the vibratory ground motion for the STP site.

*Quaternary Growth Faults*

In FSAR Subsection 2.5S.1.1.4.4.5.4, the applicant stated that although evidence exists to support Quaternary deformation on growth faults in the STP site region, no new information published since the 1986, EPRI source model studies necessitates an update to the source models. In addition, these growth faults are understood to be confined to the overlying coastal plain section and do not penetrate the crystalline basement rock. Therefore, the applicant implied that these faults do not have the ability to generate significant earthquakes. The applicant concluded that gulf coastal growth faults are adequately accounted for in the EPRI seismic source models and no updates are required.

The staff reviewed FSAR Subsection 2.5S.1.1.4.4.5.4, and concluded that the applicant has adequately characterized the evidence for Quaternary deformation on growth faults in the site region. The applicant provided a more detailed description of growth faults as they relate to the STP site in FSAR Subsection 2.5S.1.2.4.2. The staff's evaluation of growth faults in the site area is included in Section 2.5S.1.4.2 of this SER. In accordance with RG 1.208, although growth faults may cause surface deformation, they are not considered capable tectonic structures and are unlikely to generate damaging earthquakes.

Staff Conclusions of the Regional Tectonic Setting

The staff reviewed FSAR Section 2.5S.1.1.4, and concluded that the applicant has provided a complete and accurate description of the regional tectonics surrounding the STP site including the tectonic history, regional tectonic stresses, gravity and magnetic signatures, and major regional tectonic structures. The staff concluded that the regional tectonic description in STP, Units 3 and 4, COL FSAR Subsection 2.5S.1.1.4, accurately reflects the current literature and state of knowledge. The applicant's description thus meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

Staff Conclusions of the Regional Geologic Description

The staff reviewed FSAR Subsection 2.5S.1.1, and concluded that the applicant has provided a complete and accurate characterization of the regional physiography, geomorphology, geologic history, stratigraphy, and tectonic setting within the 320-km (200-mi) radius of the STP site. The applicant's description thus meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

2.5S.1.4.2 Site Area Geology

FSAR Subsection 2.5S.1.2, discusses the local site area geology as well as the site engineering geology, including the effects of human activities on the site area. The following discusses the staff's evaluation of the applicant's information in FSAR Subsection 2.5S.1.2, and in the applicant's responses to the staff's RAIs.

**Site Area Physiography and Geomorphology**

The STP site lies within the Coastal Prairies subprovince of the Gulf Coastal Plains Physiographic Province. As shown in SER Figure 2.5S.1-6 (reproduced from FSAR Figure 2.5S.1-27, "Site Area Geological Map (5-mile radius)," sands and clays of the Pleistocene-age Beaumont Formation extend across the entire site area and make up a majority of the surficial sediments. In addition, Holocene-age alluvial sediments overlie the Beaumont strata in a small portion of the eastern site area adjacent to the Colorado River. The staff reviewed STP COL FSAR Subsection 2.5S.1.2.1, and concluded that the applicant has provided an adequate description of the site's physiography and geomorphology in support of the STP COL application.

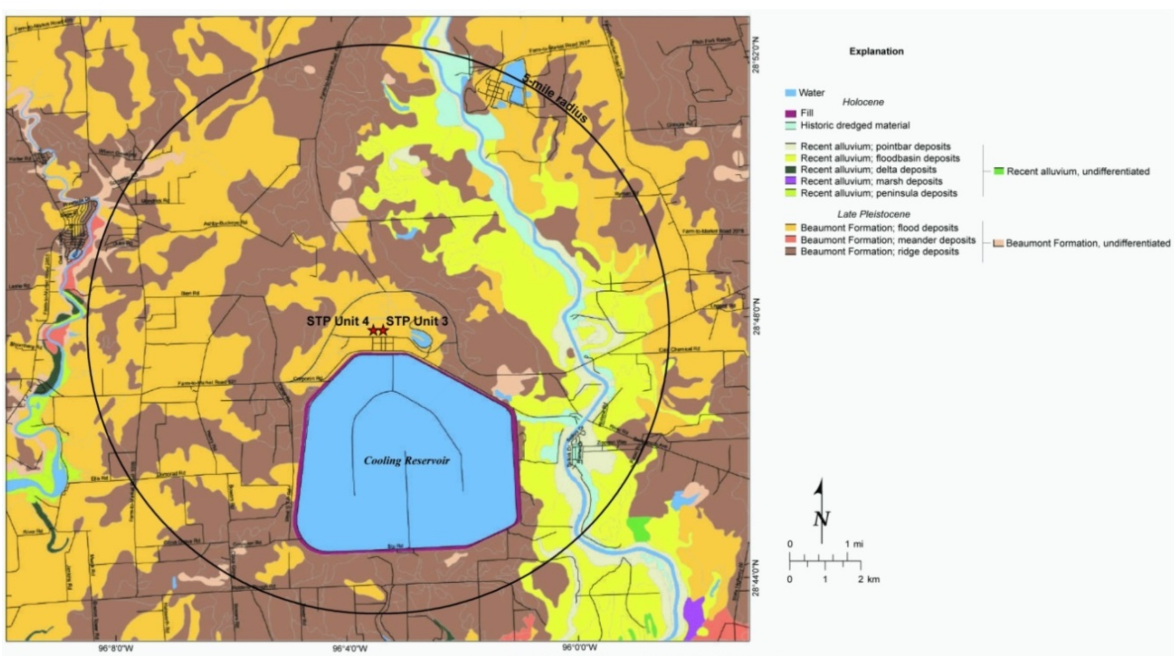


**Figure 2.5S.1-6 Geologic Map of the STP Site Area (FSAR Figure 2.5S.1-27)**

**Site Area Geologic History**

FSAR Subsection 2.5S.1.2.2 describes the geologic history of the site area, including major tectonic events that took place before the Cenozoic Era and four Pleistocene-age glacial cycles. The applicant stated that the Quaternary Period was dominated by an almost continuous sedimentary deposition that led to the gulfward migration of the shoreline. The staff reviewed FSAR Subsection 2.5S.1.2.2, and concluded that the applicant has provided a thorough and adequate description of the site's geologic history in support of the STP COL application.

**Site Area Stratigraphy**

FSAR Subsection 2.5S.1.2.3, describes the stratigraphic units that underlie the STP site area, including the Pleistocene Beaumont Formation and, in a few places, Holocene alluvial deposits. The sands and clays of the Beaumont Formation were deposited by ancestral streams of the Colorado River. The Colorado River is located approximately 5.5 km (about 3.5 mi) east of the STP site. The applicant stated that the Beaumont formation is approximately 122 m (400 ft) thick beneath the site. FSAR Subsection 2.5S.1.2.3, describes 12 strata of the Beaumont Formation based on material properties identified during subsurface investigations for STP, Units 3 and 4. FSAR Section 2.5S.4, includes a more detailed description of the subsurface investigations at the STP site, and Section 2.5S.4 of this SER includes the staff's evaluation of the applicant's subsurface investigations. The staff reviewed FSAR Subsection 2.5S.1.2.3 and concluded that the applicant has provided a thorough and adequate description of the site area stratigraphy in support of the STP, Units 3 and 4, COL application.

**Site Area Geologic Structures**

FSAR Subsection 2.5S.1.2.4, describes geologic structures in the site area, including basement structures and growth faults. The continental crust that makes up the basement rock is interpreted to be "thin transitional crust" (i.e., a portion of the crust that was exposed to considerable extension but not necessarily to actual rifting). With regard to "discrete" faults or structures in the basement rock, the applicant concluded that no new information developed about such structures has emerged since the 1986, EPRI studies. The applicant stated that buried growth faults are the only geologic structures in the STP site area. However, the applicant concluded that no growth faults project through the site location (defined as the 1-km [0.6-mi] site radius) or through the STP, Units 3 and 4, "footprint."

The applicant describes previous growth fault investigations for STP, Units 1 and 2, (STPEGS, 2006) as well as more recent investigations that include those conducted for the STP, Units 3 and 4, COL investigations. Regarding the earlier STP, Units 1 and 2, investigations, the applicant describes 10 growth faults in the site area. Seismic reflection data indicate that seven of these faults are overlain by at least 1.5 km (5,000 ft) of undisturbed sediments and therefore, based on stratigraphic correlations, those faults have not been active since at least the Miocene Epoch (i.e., the faults are older than 5.3 Ma). Two of the ten growth faults, "A" and "I," may approach within 300 m (1,000 ft) of the ground surface in the STP site area. However, the UFSAR for STP, Units 1 and 2, concluded that there is no evidence for growth faults "A" or "I" at the ground surface. This conclusion is based on subsurface data, remote sensing imagery, and undeformed strata exposed in an excavated channel where growth fault "I" is inferred to project.

As part of the growth fault investigation for STP, Units 3 and 4, the applicant compiled data from seven sources, including the UFSAR for STP, Units 1 and 2. The applicant stated that there is uncertainty on the order of several miles, which is associated with projecting growth faults at depth to the surface. In addition, the resolution limits associated with some of the data do not allow the growth faults to be identified in the shallow surface or even at depths of less than approximately 1.8 km (6,000 ft) beneath the surface. The applicant stated in FSAR Subsection 2.5S.1.2, that "the most detailed subsurface mapping of growth faults in the site area remains the work documented in the UFSAR for STP 1 & 2."

The applicant performed new aerial photographic analyses as well as aerial and field reconnaissance investigations (including lineament analyses) to evaluate growth faults in the STP site area for the COL investigation. The applicant focused these investigations on growth faults "A" and "I" (Matagorda STP 12I/Matagorda GMO), which were identified in the UFSAR for STP, Units 1 and 2, because their inferred surface projections lie within the STP site area, and because there is evidence that they may deform strata younger than 5.3 Ma.

The applicant examined linear features in the STP site area and investigated spatial associations between these features and the inferred surface projections of known growth faults. In conclusion, the applicant found that distinct linear features are associated with the surface projections for growth fault Matagorda STP 12I, but these features are less pronounced or nonexistent for other growth faults in the site area. The applicant conducted an aerial reconnaissance to investigate linear and topographic features from above but found no evidence for such features within the 8-km (5-mile) site radius.

The applicant surveyed four topographic profiles across the surface projection of growth fault Matagorda STP12I (coincident with growth fault Matagorda GMO, identified after the UFSAR for STP, Units 1 and 2). The applicant chose to perform the topographic surveys across growth fault STP 12I because the applicant had identified linear slope breaks associated with this fault in its aerial and field reconnaissance investigations. The applicant also prepared and evaluated an east-west cross section of correlated borehole data for the southern limits of the cooling reservoir to assess potential offsets across the projection of growth fault Matagorda GMO.

The staff reviewed FSAR Subsection 2.5S.1.2.4.2 and issued a number of RAIs related to growth faults in the STP site area. Those RAIs are discussed below.

Given that seismic reflection, well log, and imagery data sources described in FSAR Subsection 2.5S.1.2.4.2.2, could not resolve surface locations of growth faults in the STP site area, the staff issued RAI 02.05.01-8, requesting the applicant to explain why the investigations for the STP COL application site do not include a light detection and ranging (LiDAR) survey of the site area to reassess evidence for possible subtle surface folding or faulting along growth faults. In its response to RAI 02.05.01-8, dated July 16, 2008 (ML082030326), the applicant updated FSAR Figures 2.5S.1-44, "Lineaments and Aerial Photographs," and 2.5S.1-45, "Growth Fault Projections and Lineaments," to include the locations of all "anomalous geomorphic features" that might indicate surficial deformation due to growth faulting. The applicant stated that growth fault Matagorda STP12I (Matagorda GMO) is the only growth fault that could be correlated with linear geomorphic features identified in the applicant's aerial photographic analysis. Therefore, it is "the only fault within the site area with a geomorphic expression of potential Quaternary activity." With regard to applying other methods such as the LiDAR to investigate potential surface displacements, the applicant stated the following:

> As presented in Subsection 2.5S.1.2.4.2.2.2 and apparent in Figure 2.5S.1-45, the monoclinal folding, lineations, and surface projections associated with fault I/GMO are strongly correlated, suggesting that the diversity of methods used to identify growth faults with surface expression (e.g., ground reconnaissance, fault projections, aerial photo analysis) were robust and capable of identifying the surface expression of growth faults if present. The robust nature of these methods then provides confidence that the methods are capable of identifying surface deformation from growth faulting throughout the site area if it exists. Therefore, it was deemed unnecessary to conduct a separate LiDAR survey to identify surface deformation associated with growth faulting.

The staff reviewed the applicant's response to RAI 02.05.01-8, including enhanced FSAR Figures 2.5S.1-44 and 2.5S.1-45, and acknowledged that the applicant has adequately identified a range of geomorphic features using available satellite imagery and aerial photographic data. In addition, the staff noted that the applicant has performed additional investigations of potential surficial features as part of aerial and field reconnaissance efforts.

In RAI 02.05.01-12, the staff requested the applicant to describe geologic processes that have the potential to influence the preservation of evidence of growth faults at the earth's surface. In its response to RAI 02.05.01-12, the applicant explained that the stratigraphic units exposed at the surface in the STP site area are most likely 100,000 to 150,000 years old. The applicant stated that surface deformations associated with growth faults near the site is often several kilometers in length. Growth faults exposed at the surface are likely expressed as broad monoclinal folds that produce gentle changes in the surface gradient. The applicant evaluated the surficial sedimentary units in the site area associated with the Beaumont Formation. The applicant explained that these relatively old surface sediments (100,000 to 150,000 years old) do not show evidence of significant soil erosion or deposition during the past "tens of thousands to hundreds of thousands of years." The applicant concluded that minimal geologic processes acting on the surface sediment strongly suggest that geologic evidence for growth faulting at the surface would persist for a long time—tens of thousands to hundreds of thousands of years.

The staff reviewed the applicant's response to RAI 02.05.01-12, and found that the applicant has adequately evaluated the potential for geologic surface processes to influence the preservation of growth faults at the earth's surface. Based on the applicant's information in FSAR Subsection 2.5S.1.2.4 and in the response to RAI 02.05.01-12, the staff concluded that the rate of erosion and deposition in the STP site area is unlikely to erase evidence of recent growth faulting. The staff acknowledged that any geologic evidence of growth faults that have ruptured the surface in at least the past tens of thousands of years should remain preserved at the surface. Therefore, RAI 02.05.01-12 is resolved and closed.

The staff also noted that the applicant's growth fault analysis and conclusions rely heavily on investigations completed for STP, Units 1 and 2. However, FSAR Subsection 2.5S.1.2.4.2 includes only limited details of those previous studies. Therefore, the staff issued RAI 02.05.01-7, requesting the applicant to provide a detailed summary of earlier investigations directed at assessing Quaternary growth faults near the STP site. In its response to RAI 02.05.01-7, dated October 1, 2008 (ML082770138), the applicant provided a comprehensive summary of the investigations and analyses completed for STP, Units 1 and 2. The applicant's summary includes key figures that identify seismic reflection line and well log locations, as well as the identified fault plane locations. The applicant's response also described the trenching and excavation studies completed for STP, Units 1 and 2, including an investigation to look for evidence of deformation above the projection of growth fault "I" (Matagorda STP 12I) in an excavated channel known as the Relocated Little Robbins Slough on the west side of the cooling reservoir for STP, Units 1 and 2. Based on the applicant's response to RAI 02.05.01-7, the staff was able to more completely evaluate the applicant's conclusions regarding growth faulting and the potential for surface deformation due to growth faults in the STP site area. Therefore, RAI 02.05.01-7 and RAI 02.05.01-8 are resolved and closed.

The staff issued multiple RAIs related to the applicant's evaluation of growth fault Matagorda STP12I (Matagorda GMO). In RAI 02.05.01-9, the staff asked the applicant: (1) to explain why the evaluation only measured the topographic offset at four locations along the surface projection of growth fault Matagorda STP 12I (Matagorda GMO); and (2) to discuss the uncertainties in projecting this growth fault to the surface. In its response to RAI 02.05.01-9, dated October 1, 2008, the applicant stated that the locations of the topographic surveys (as shown in SER Figure 2.5S.1-5) are based on evidence for monoclinal folding that the applicant observed during initial field reconnaissance studies. The applicant's survey did not extend beyond the westernmost observed folding but did include one location (STP L4) that was along strike with the folding and was the closest available location to the cooling reservoir. The applicant stated that even though there are uncertainties in the projection of growth fault Matagorda STP 12I (Matagorda GMO), the fact that the monoclinal folding is evident in three of the four survey profiles provides confidence in the applicant's "best estimate" projection of Matagorda STP 12I (Matagorda GMO).

The staff reviewed the applicant's response to RAI 02.05.01-9. The staff concurred with the applicant's reasoning for selecting profile locations STP L1 through STP L4 based on the evidence for monoclinal folding at the surface and on the applicant's preferred surface projection for growth fault STP GMO. The applicant's response assumes that the surface projection of growth fault Matagorda STP 12I (Matagorda GMO) bends to the southeast around the cooling reservoir (as shown in SER Figures 2.5S.1-4 and 2.5S.1-5) rather than through the reservoir.

In RAI 02.05.01-10, the staff asked the applicant to explain the inference that the surface projection of fault GMO/Matagorda 12I bends to the southeast around the reservoir and not through it. In addition, the staff asked the applicant to discuss whether the methods used to measure possible cumulative displacement across the projection of fault GMO/Matagorda 12I (as described in FSAR Subsection 2.5S.1.2.4.2.2.2) are capable of measuring displacements over hundreds of years, and whether surface displacements such as those associated with fault GMO/Matagorda 12I can be preserved at the surface for hundreds to thousands of years.

In its response to RAI 2.5.1-10, dated September 4, 2008 (ML082530449), the applicant stated that the surface projection of growth fault GMO to the southeast (around the reservoir) was developed by the Geomap Company and reflects data the company provided to the applicant, as referenced in the FSAR (Geomap, 2007). The surface projection also applies only to fault Matagorda GMO and not to Matagorda STP 12I. With regard to deformation in the form of monoclinal folding associated with GMO/Matagorda STP 12I, the applicant stated that the methods used to measure structural relief across the projection of growth fault Matagorda STP 12I (Matagorda GMO) are reliable over long periods of time, given that "there has been very little erosional or depositional modification of the land surface within the last 10,000 years." Therefore, the applicant concluded that it is "unlikely that surface processes would completely mask or degrade the increases in structural relief of the hypothetical monoclinal folding" on the order of tens of centimeters over hundreds of years. In addition, the applicant noted that "well-developed and mature" soils are present in the site area and these soils developed over thousands to tens of thousands of years. These soils indicate that typical surface processes in the site area are "minimal," occur slowly over long periods of time, and are therefore not likely to obliterate broad monoclinal folds at the surface.

Based on the applicant's response to RAI 02.05.01-10, the staff concluded that the applicant has provided detailed descriptions of the surficial processes affecting the STP site area. The staff concurred with the applicant that surficial deformation due to growth faulting in the STP site area is not likely to be removed or masked by surficial processes over periods of less than thousands of years, given the lack of notable erosion and deposition currently observed for the site area. With respect to the surface projection of growth fault Matagorda STP 12I (Matagorda GMO), the staff reviewed the applicant's response to RAI 02.05.01-10. The staff acknowledged that the surface projection only refers to growth fault Matagorda GMO and does not reflect the applicant's inferred surface projection for growth fault Matagorda STP12I. The staff noted that SER Figure 2.5S.1-5 (based on FSAR Figure 2.5S.1-45, "Growth Fault Projections and Lineaments"), shows linear features within the cooling reservoir that represent slope breaks and vegetation lineaments along strike with growth fault Matagorda STP 12I (Matagorda GMO). The staff issued RAI 02.05.01-20, requesting the applicant to evaluate whether these linear features represent a northeast extension of growth fault Matagorda STP 12I (Matagorda GMO), which is different from the projection shown in SER Figures 2.5S.1-4 and 2.5S.1-5.

In its response to RAI 02.05.01-20, dated July 20, 2009 (ML092030132), the applicant stated the finding that the linear features are "identified from preconstruction aerial photographs" that extend beneath the cooling reservoir and are not associated with growth fault GMO. The applicant's determination is based on the fact that a majority of the linear features are vegetation lineaments (and not slope breaks). The applicant therefore did not feel that the GMO would project to the location of the observed linear features. Based on projected uncertainty bounds for the projection of growth fault GMO, the applicant concluded that "the projection of GMO, including its expected uncertainty, is well south of the lineaments within the cooling reservoir" and the applicant "considers it very unlikely that the lineaments within the reservoir are related to growth fault GMO." The applicant also notes that the broad monoclinal folding associated with growth fault Matagorda STP I21, adjacent to the cooling reservoir, does not appear to extend into the cooling reservoir. This finding is based on the applicant's four topographic surveys along this growth fault and on the applicant's field observations.

The staff reviewed the applicant's response to RAI 02.05.01-20, and acknowledged that the applicant has performed a robust investigation of growth fault Matagorda STP 12I/GMO based on the applicant's inferred surface projection of growth fault STP 12I/GMO. Furthermore, the staff acknowledged that the linear features identified within the cooling reservoir in SER Figure 2.5S.1-5 (FSAR Figure 2.5.1-45) are now covered and therefore, the applicant cannot investigate them further. The staff concluded that the applicant's response to RAI 02.05.01-20, provides an acceptable evaluation of the projected trace of STP 12I/GMO given the lack of more convincing field observations, and the inability to further evaluate the linear features beneath the cooling reservoir. Therefore, RAI 02.05.01-9, RAI 02.05.01-10, and RAI 02.05.01-20, are resolved and closed.

The staff noted that FSAR Figure 2.5S.1-43, "Site Vicinity (5-mile radius) Growth Fault Surface Projections," shows multiple northeast-southwest trending faults (identified in the Geomap data) that project into the STP site area. Because these faults are not discussed in detail in the FSAR, the staff issued RAI 02.05.01-13, requesting the applicant: (1) to provide an additional explanation of the Geomap faults that project into the STP site area; and (2) to justify whether any of these faults project through the proposed STP, Units 3 and 4, or through the cooling reservoir.

In its response to RAI 02.05.01-13, dated August 12, 2008 (ML082270381), the applicant provided additional descriptions of four growth faults identified in the Geomap data: GMH, GMI, GMK, and GML. None of these four faults extends to within 2.1 km (7,000 ft) of the ground surface. The Geomap interpretations were based mostly on well log data, and little well log data are available near the site that could constrain the locations of these faults at the site. The applicant stated that the Geomap data cannot be correlated with the seismic reflection data obtained for STP, Units 1 and 2, "because none of the seismic reflection lines from the UFSAR cross the Geomap growth fault traces." The shallow seismic reflection data collected for STP, Units 1 and 2, provide the most useful information on the subsurface strata closest to the STP site. These data show no evidence for deformation of Miocene and younger (less than 5.3 Ma) sedimentary units beneath the site. In addition, the applicant's aerial and field reconnaissance and subsequent field investigations for this COL application found no evidence for surface displacement due to growth faulting at the proposed STP, Units 3 and 4, site.

The staff reviewed the applicant's response to RAI 02.05.01-13, and acknowledged that the applicant has performed a robust investigation to evaluate the potential for growth faulting beneath the proposed STP, Units 3 and 4, site. However, the staff was concerned that the seismic reflection data may not be appropriate for evaluating growth fault deformation within the Quaternary units directly beneath the site (i.e., within the upper 120 m [400 ft]). Therefore, the staff issued RAI 02.05.01-21, requesting the applicant to describe the resolution limits associated with the data used to interpret growth faulting, or the lack of growth faulting, within the Quaternary units beneath the STP site.

In its response to RAI 02.05.01-21, dated July 20, 2009 (ML092030132), the applicant provided the two methods the applicant relied on to evaluate the presence or absence of growth faulting in the Quaternary units beneath the STP site: (1) the analysis and interpretation of subsurface data (including seismic reflection data); and (2) surface investigations (including aerial photo interpretation, aerial and field reconnaissance, and field investigations).

With respect to the subsurface investigations conducted for the existing STP, Units 1 and 2, the applicant stated that the UFSAR for STP, Units 1 and 2, did not discuss resolution limits associated with the seismic reflection data. However, the seismic reflection data identify two growth faults, STP12A and STP 12I, which approach within 275 m (900 ft) of the earth's surface beneath the STP site area. The applicant also stated that shallow growth faults typically "sole into deeper growth fault systems." Therefore, the applicant concluded that if shallow growth faults do exist, they should be rooted in deeper structures and therefore should produce a signature in the data. In addition, the applicant noted that "growth faults tend to have greater offsets downdip along their fault plane because the updip portions of the fault are younger and have experienced less dip." Based on this information, the applicant concluded that "growth faults with small offsets at shallow depth should be easier to identify at greater depths where they will have larger offsets."

The applicant also relied on multiple aerial and field reconnaissance and subsequent field investigations to confirm the presence or absence of deformation due to growth faulting of the near surface. The applicant stated that the only evidence of growth faulting in the STP site area is "broad, monoclinal folding and tilting" associated with growth fault Matagorda STP 12I and

Matagorda GMO, as previously described. The applicant also refered to its response to RAI 02.05.01-10, which concluded that: (1) deformation due to growth faulting of the Pleistocene-age Beaumont Formation underlying the site should be preserved and “presently observable”; and (2) the lack of deformation observable at the surface should indicate a lack of deformation associated with the Beaumont Formation.

The staff reviewed the response to RAI 02.05.01-21. The staff concluded that the applicant has reasonably justified the applicability of the seismic reflection data to resolve shallow growth fault structures, assuming that they are deeply rooted in a deeper detachment horizon. To supplement the seismic reflection data, the staff concluded that the applicant has adequately incorporated a range of methods for evaluating deformation at the surface, if the deformation is not resolved using the subsurface data. Furthermore, based on the combination of the shallow seismic reflection data for STP, Units 1 and 2, and the results of the applicant’s recent field investigations, the staff concluded that the applicant’s assessment that no shallow growth faults displace the Quaternary strata beneath the STP site is reasonable and adequately justified. Therefore, RAI 02.05.01-13 and RAI 02.05.01-21, are resolved and closed.

FSAR Subsection 2.5S.1.2.4.3, states that fault GMP: (1) extends beneath the cooling reservoir, (2) is the closest growth fault to STP, Units 3 and 4, and (3) has a surface projection approximately 2.25 km (1.4 mi) from the proposed STP, Units 3 and 4. However, the applicant does not provide any additional details about growth fault GMP in the COL application. This is the closest growth fault to the STP site and it was not previously characterized in the FSAR for STP, Units 1 and 2. Therefore, the staff issued RAI 02.05.01-19, requesting the applicant to describe growth fault GMP more thoroughly, including any additional investigations that the applicant performed to evaluate this fault. In its response to RAI 02.05.01-19, dated July 20, 2009 (ML092030132), the applicant stated that the surface projection of growth fault GMP (as indicated in SER Figure 2.5S.1-5) appears to trend northwest through the cooling reservoir and just to the west of the proposed STP, Units 3 and 4. The applicant stated that the “perceived trend based on the surface projection does not represent the actual trend of the growth fault at depth” and that the Geomap structural contour maps indicate that growth fault GMP “trends to the west, subparallel to the surface projection of growth fault GMO and not to the north towards the STP 3 & 4 site.” Finally, the applicant stated that “the contrast in trend of the surface projection to the trace of the fault at depth is due to limitations associated with developing the growth fault surface projections.” The applicant concluded that based on the seismic reflection data originally evaluated for STP, Units 1 and 2, growth fault GMP does not pose a deformation hazard at the site and that the seismic reflection data originally evaluated for STP, Units 1 and 2, supports this conclusion.

The staff reviewed the applicant’s response to RAI 02.05.01-19, and concluded that the applicant’s response to RAI 02.05.01-19, which states that growth fault GMP actually trends to the west, conflicts with the stated information in FSAR Subsection 2.5S.1.2.4.3 (in both Revision 3 of the FSAR and as revised in the response to RAI 02.05.01-19). Therefore, the staff issued RAI 02.05.01-22, requesting the applicant to resolve the inconsistencies in FSAR Subsection 2.5S.1.2.4.3 (Revision 3) regarding the projection of growth fault GMP within the STP site area. In its response to RAI 02.05.02-22, dated March 10, 2010 (ML100620824), the applicant stated that the “perceived inconsistency” between FSAR Subsection 2.5S.1.2.4.3 (Revision 3) and FSAR Figures 2.5S.1-42, 2.5S.1-43 and 2.5S.1-45 (Revision 3), is due to the level of detail provided in the FSAR to document the depiction of the surface projection of growth fault GMP in the FSAR Figures. Therefore, in its response to RAI 02.05.01-22, the applicant proposed to modify FSAR Subsection 2.5S.1.2.4.3

to “include a discussion of how the Geomap data demonstrates the change in the strike of GMP.” In its response, the applicant indicated that the Geomap data used to constrain the trend of growth fault GMP relied on structural contour maps of two horizons at depth. The lower horizon depicts growth fault GMP having a northwest trend (toward STP, Units 3 and 4,) and extending for approximately 1.6 km (1 mi). The upper horizon, however, depicts growth fault GMP having a westward trend and extending for approximately 4.8 km (3 mi) beyond the extent of the STP, Units 3 and 4, site. The applicant stated that while the lower horizon dictates the surface projection of growth fault GMP it does not accurately reflect the true westward trend of the GMP fault, which the applicant is confident does not trend toward STP, Units 3 and 4.

The staff reviewed the applicant’s responses to RAI 02.05.01-19, and RAI 02.05.01-22, and acknowledged that the applicant has adequately evaluated growth fault GMP given the limited availability of data. The staff concluded that the information contained in the uppermost horizon of the Geomap data would likely reflect the most accurate trend of growth fault GMP given a larger number of data points within the upper horizon as well as the evidence that growth fault GMP extends to the west for a longer distance. In addition, the staff concluded that the westward trend of growth fault GMP is consistent with the overall local growth fault trend. In the response to RAI 02.05.01-22, the applicant proposed to revise FSAR Subsection 2.5S.1.2.4.3. The staff confirmed that the proposed changes to Subsection 2.5S.1.2.4.3 are included in COL FSAR Revision 4. Therefore, RAI 02.05.01-22 is resolved and closed.

Staff Conclusions Regarding Site Area Geologic Structures

The staff reviewed FSAR Subsection 2.5S.1.2.4.1, and concluded that the applicant has provided a complete and accurate description of the geologic structures in the STP site area. In addition, the staff concluded that the applicant has adequately characterized the geologic structures (specifically growth faults) in the STP site area in support of the STP COL application. Finally, the staff concluded that the description of site area geologic structures in STP COL FSAR Subsection 2.5S.1.2.4.1, meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

**Site Area Geologic Hazard Evaluation**

FSAR Subsection 2.5S.1.2.5, discusses geologic hazards at the STP, Units 3 and 4, site. The applicant concluded that there is no evidence for dissolution, zones of deformation, or volcanic activity in the STP site area. The staff reviewed STP COL FSAR Subsection 2.5S.1.2.5 and concluded that based on the available literature and geologic data for the site; there is no evidence that geologic hazards have impacted the STP site area. The applicant appropriately discussed the evidence for dissolution features and volcanic activity, neither of which is known to have occurred at the STP site at least within the past two million years. The applicant does not evaluate the earthquake hazard potential in FSAR Subsection 2.5S.1.2.5. However, the applicant’s seismic hazard analysis is discussed in detail in FSAR Section 2.5S.2, and the staff’s evaluation is in Section 2.5S.2 of this SER. The applicant also does not discuss deformation due to growth faulting in FSAR Subsection 2.5S.1.2.5. However, growth faults are discussed in other parts of FSAR Section 2.5S.1 and in FSAR Section 2.5S.3. The staff’s evaluation of growth faulting in the STP site area is in SER Subsection 2.5S.3.4.2.

**Site Engineering Geology Evaluation**

FSAR Subsection 2.5S.1.2.6, discusses the applicant’s evaluation of the site engineering geology, including potential effects of human activities at the STP site. The applicant stated that

FSAR Section 2.5S.4, discusses engineering soil properties and the behavior of foundation materials. The applicant concluded that there is no evidence of weathering or dissolution at the STP site and there are no deformational zones, capable tectonic structures, or evidence of previous earthquake activity at the STP site. The applicant will conduct excavation mapping during construction to evaluate any potential features beneath the site.

Prior Earthquake Effects

In FSAR Subsection 2.5S.1.2.6.4, the applicant stated that outcrops were examined in the STP site area as part of its geologic field investigation for STP, Units 3 and 4. Previous FSAR sections describe the abundant late Pleistocene and Holocene alluvial deposits that overlie buried Mesozoic structures within thin transitional crust at the STP site. The staff noted that these geologic conditions are similar to conditions in other parts of the CEUS where researchers have used earthquake-induced liquefaction features (which are preserved in the sedimentary record) to estimate timing, source areas, magnitudes, and recurrence intervals of large prehistoric earthquakes. Holocene and Late Pleistocene fluvial deposits that are likely to be susceptible to liquefaction during large earthquakes occur in the STP site area and site vicinity. The staff therefore issued RAI 02.05.01-15 requesting the applicant to explain why there was no effort to search for liquefaction features potentially produced during large earthquakes near the STP site.

In its response to RAI 02.05.01-15, dated July 16, 2008 (ML082030326), the applicant identified an extensive literature review for the STP, Units 3 and 4, COL application that looked for but did not find any reports of previously identified liquefaction features in the site region. In addition, the applicant stated that there is no record of moderate to large earthquakes in the site region. However, the applicant did conduct a paleoliquefaction investigation "within the greater site area" that included an aerial photographic analysis and field reconnaissance. The applicant discussed this aerial photographic analysis in FSAR Subsection 2.5S.1.2.4.2.2.2 with respect to growth fault investigations and refers to any potential paleoliquefaction features as "potentially anomalous geomorphic features." The applicant concluded that "none of these features provided evidence of liquefaction." The applicant also looked for liquefaction features along more than 24 km (15 mi) of Colorado River bank exposures and found no evidence of earthquake-induced liquefaction.

The staff noted that the available literature that the applicant describes in the response to RAI 02.05.01-15, does not clearly identify whether or not liquefaction investigations have even been conducted in the area surrounding the STP site. In addition, a lack of moderate to large magnitude historical earthquakes does not preclude the occurrence of prehistoric earthquakes of a similar magnitude in an area, which thus explains the reliance on paleoliquefaction investigations. The staff acknowledged that the applicant has thoroughly examined aerial photographs for evidence of paleoliquefaction features. However, a majority of earthquake-induced liquefaction features are not identifiable on aerial photographs due to the size of the features, soil mixing, vegetative cover, and the fact that they may not be exposed at the ground surface. Therefore, the staff focused its review of the response to RAI 02.05.01-15, on the applicant's field reconnaissance investigation.

Because the applicant provided little description of its paleoliquefaction field investigation in response to RAI 02.05.01-15, other than to say that the applicant examined over 24 km (15 mi) of exposed riverbank and found no evidence of liquefaction, the staff issued RAI 02.05.01-18, requesting the applicant to provide more details specific to this investigation. In its response to

RAI 02.5.01-018, dated July 20, 2009 (ML092030132), the applicant provided a detailed description of the STP, Units 3 and 4, site paleoliquefaction investigations including: (1) the quality of the riverbank exposures along the Colorado River, (2) the sedimentary conditions at the locations investigated, and (3) the types of earthquake-related features that the applicant looked for in the exposures. The response stated that most of the riverbank that the applicant investigated along the Colorado River provided good exposure to look for sedimentary features. The applicant stated that sedimentary conditions, including the availability of laterally continuous coarse sands overlain by a laterally continuous cap of fine-grained silts, and ground water conditions along the Colorado River are favorable for liquefaction to occur. However, the applicant found no evidence in the riverbank exposures to indicate that horizontal sedimentary layers were disturbed due to subsurface liquefaction or earthquake-induced lateral spreading. The only deformation of the riverbank exposures that the applicant discovered was recent slumping of riverbank material likely due to lateral erosion. Finally, the applicant's efforts to investigate smaller streams and tributaries of the Colorado River found that most of these secondary routes were heavily vegetated or inaccessible. Where good exposures did exist, the applicant found no evidence of paleoliquefaction in the exposed deposits.

Based on the level of detail that the applicant provides in response to RAI 02.05.01-18, the staff concluded that the applicant has conducted adequate investigations of riverbank exposures in the STP site area to evaluate the presence or absence of liquefaction features. Furthermore, the staff found that the applicant has adequately characterized the sedimentary units adjacent to the Colorado River and has adequately justified its conclusion that liquefaction features are not evident in the riverbank sections investigated for STP, Units 3 and 4. In its response to RAI 02.05.01-18, the applicant also proposed to update FSAR Subsection 2.5S.1.2.6.4, with a more detailed description of the investigations for prior earthquake effects at the STP site. The staff confirmed that the proposed change to Subsection 2.5S.1.2.6.4 is included in COL FSAR Revision 4. Therefore, this issue in RAI 02.05.01-15 and RAI 02.05.01-18, is resolved and closed.

Effects of Human Activities

FSAR Subsection 2.5S.1.2.6.5, discusses the effects of human activities at the STP site, specifically the effects of oil and ground water withdrawal that could lead to subsidence of the underlying sedimentary units. The applicant calculated the anticipated maximum subsidence at the STP site due to construction dewatering and concluded that the calculated values of 1.2 to 1.5 cm (0.04 to 0.05 ft) are not likely, because some of the extracted water will be replaced by storm water or runoff. The applicant stated that no mining or "excessive" ground water injection takes place in the STP site area. The applicant discussed ground water conditions and the effects of human activities related to ground water in more detail in FSAR Section 2.4S.12. The staff's ground water evaluation is in Section 2.4S.12 of this SER.

The staff issued RAI 02.05.01-14, requesting the applicant to describe the potential for future subsidence due to human activities (such as fluid and gas injection or withdrawal) and effects from these activities that include differential displacement across growth faults near the STP cooling reservoir. In its response to RAI 02.05.01-14, dated August 27, 2008 (ML082490086), the applicant stated that growth fault Matagorda GMO (STP 12I) is the only known fault that approaches within 1.5 km (5,000 ft) of the ground surface and that: (1) shows potential Quaternary displacement, and (2) projects to within 3.2 km (2 mi) of the STP cooling reservoir. The applicant described fluid withdrawal activities associated with the Chicot aquifer and hydrocarbon production activities near the STP site. The applicant stated that the UFSAR for the existing STP, Units 1 and 2, does not document any evidence of differential subsidence due to fluid extraction or other human activities in the STP site area through the early 1980s. Based on production records from the Texas Railroad Commission (Texas RRC, 2008), the applicant stated that hydrocarbon production closest to the STP site is considerably less than production before the construction of the existing STP, Units 1 and 2. Finally, the applicant's analysis of aerial photographs taken since 1958 (before, during, and after construction of STP, Units 1 and 2,) finds "no noticeable surface deformation from movement on growth fault GMO/STP 12I for at least the last 50 years." Given the evidence presented above, the applicant concluded that "it is highly unlikely" that subsidence due to fluid withdrawal or hydrocarbon production will cause displacement across faults near the STP cooling reservoir."

The staff reviewed the response to RAI 02.05.01-14, and concurred with the applicant's conclusion that deformation across growth faults due to human activities near the cooling reservoir is unlikely. Furthermore, the staff concluded that the applicant has adequately investigated records of hydrocarbon production and fluid withdrawal during the past 20 to 25 years, in order to fully evaluate the potential for future deformation across any growth faults beneath the cooling reservoir due to these activities. Therefore, RAI 02.05.01-14 is resolved and closed.

Staff Conclusions Regarding the Site Engineering Geology Evaluation

The staff reviewed FSAR Subsection 2.5S.1.2.6, and concluded that the applicant has provided an adequate description of the site engineering geology for the STP site to address COL License information Item 2.23. The applicant stated in FSAR Subsection 2.5S.1.2.6, that "excavation mapping and evaluation is required during construction." Based on the fact that numerous growth faults are known to occur within the STP site vicinity and that at least one growth fault is believed to project within the STP site area, the staff expressed concern regarding the potential for previously unmapped growth faults that may exist immediately beneath the proposed STP, Units 3 and 4, to cause deformation at the site. Therefore, the staff issued RAI 02.05.01-23, requesting the applicant to provide a commitment to: (1) perform geologic mapping (based on the guidance in RG 1.208) of future excavations for safety-related structures; (2) evaluate any geologic features that are encountered; and (3) notify the NRC once any excavations for safety-related structures are open for inspection. In its response to RAI 02.05.01-23, dated March 1, 2010 (ML100620824), the applicant stated that Subsection 3.9S.3.11, of the Environmental Report (in the COL application, Part 3) describes the applicant's plans for geologic mapping of excavations. The staff confirmed that the proposed change to the description of its plans for geologic mapping during excavations was included in COL FSAR Revision 4. Therefore, this issue in RAI 02.05.01-23 is resolved and closed.

### *2.5S.1.5 Post Combined License Activities*

The following License Condition is identified in SER Subsection 2.5S.1.4.2 as the responsibility of the COL holder:

> **License Condition 2.5.1-1**:
>
> The Licensee shall perform detailed geologic mapping of excavations for safety-related structures; examine and evaluate geologic features discovered in the excavations; and notify the Director of the Office of New Reactors, or the Director's designee, in writing,

once excavations for safety-related structures are open for examination by the NRC staff.

### *2.5S.1.6 Conclusion*

The staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information relating to the basic geologic and seismic information, and no outstanding information is expected to be addressed in the STP COL FSAR related to this subsection.

The staff found that the applicant has provided a thorough characterization of the geologic and seismic characteristics of the STP site, as required by 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii). In addition, the staff finds that the applicant has identified and appropriately characterized all seismic sources significant for determining the GMRS or site-specific SSE for the COL site, in accordance with 10 CFR 100.23 and 10 CFR 52.79 and consistent with RG 1.208. Based on the applicant's geologic investigations of the site vicinity and the site area, the staff finds that the applicant has properly characterized regional and site lithology, stratigraphy, geologic and tectonic history, and structural geology at the site, in addition to the subsurface soil and rock units. The staff also concluded that there is no potential for the effects of human activity (e.g., mining activity or ground water injection or withdrawal) to compromise the safety of the site. Therefore, the staff finds that the proposed STP COL site is acceptable from a geologic standpoint and meets the requirements of 10 CFR 100.23.

## 2.5S.2 Vibratory Ground Motion

### *2.5S.2.1 Introduction*

The evaluation of vibratory ground motion is based on seismic, geologic, geophysical, and geotechnical investigations carried out to determine the site-specific GMRS, or the SSE ground motion for the site. RG 1.208 defines the GMRS as the site-specific SSE to distinguish it from the certified seismic design response spectra (CSDRS) used as the design ground motion for the various certified designs, as well as for the foundation input response spectra (FIRS), which is the site-specific ground motion at the foundation level rather than at the surface. The development of the GMRS is based on a detailed evaluation of earthquake potential, which takes into account the regional and local geology; Quaternary tectonics; seismicity; and site-specific geotechnical engineering characteristics of the site's subsurface material. These investigations describe the seismicity of the site region and the correlation between earthquake activity and seismic sources. The applicant identified and characterized seismic sources, including the rates of occurrence of earthquakes associated with each seismic source. Seismic sources that cover any portion of the 320-km (200-mi) site radius must be identified. More distant sources that have a potential for earthquakes large enough to affect the site must also be identified. Seismic sources can be capable tectonic sources or seismogenic sources. This review covers the following specific areas: (1) seismicity, (2) geologic and tectonic characteristics of the site and region, (3) the correlation between earthquake activity and seismic sources, (4) probabilistic seismic hazard analysis and controlling earthquakes, (5) seismic wave transmission characteristics of the site, (6) site-specific GMRS; and (7) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

### *2.5S.2.2 Summary of Application*

In Section 2.5S.2 of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant provides site-specific supplemental information to address COL License Information Item 2.24 identified in DCD Tier 2, Revision 4, Section 2.3.

<u>*COL License Information Item*</u>

- COL License Information Item 2.24 Vibratory Ground Motion

This COL license information item addresses the provision for the collection of site-specific geological, seismological, and geotechnical data and the comparison of the site-specific SSE GMRS to the design response spectra.

FSAR Section 2.5S.2, describes the potential vibratory ground motion at the STP, Units 3 and 4, site. To determine whether an update of the 1989, EPRI-SOG seismic source and ground motion models was necessary, the applicant reviewed the literature published since the mid to late 1980s and performed sensitivity analyses. The applicant developed and evaluated the GMRS according to the performance-based approach recommended by RG 1.208. Based on this evaluation, the applicant presents the following vibratory ground motion information for the STP, Units 3 and 4, site.

2.5S.2.2.1 Seismicity

FSAR Subsection 2.5S.2.1, describes the development of a current earthquake catalog for the STP, Units 3 and 4, site. The applicant uses the methodology in RG 1.208 by starting with the EPRI-SOG historical earthquake catalog (EPRI NP-4726-A, 1988), which is complete from 1627 to 1984. The EPRI-SOG original seismic source models were developed for the CEUS in 1986 by the six EPRI-SOG ESTs. The applicant updated EPRI-SOG's historical earthquake catalog with seismicity from 1985 or later (through November 2006) using current seismicity catalogs, including the Advanced National Seismic System (ANSS), the International Seismological Centre (ISC), and the Preliminary Determination of Epicenters (PDE), in addition to data from various published journal articles (Stover and Coffman, 1993; Stover et al., 1984; Rinehart et al., 1982). The applicant deleted non-preferred and duplicate entries for the final updated catalog and converted the different catalog magnitude scales to body wave magnitude (mb), which is the scale used in the EPRI-SOG catalog.

The applicant's seismicity catalog update includes: (1) seismicity data from 1985 through November 2006, within the latitude-longitude window of 24° to 40° N and 107° to 83° W, which includes the 320-km (200-mi) site radius; and (2) seismicity throughout portions of the Gulf of Mexico that were not included in the original EPRI-SOG catalog, which is comprised of events that occurred between 1927 and 2006. After calculating a common $m_b$ magnitude scale and adding the updated seismicity to the original EPRI-SOG earthquake catalog, the applicant then converted all event magnitudes in the updated earthquake catalog to moment magnitude (M), a more commonly used magnitude scale.

The applicant's updated earthquake catalog within the designated latitude-longitude window (24° to 40° N and 107° to 83° W) is listed in FSAR Table 2.5S.2-3, "Seismicity Catalog from 1985 to Present for the Project Investigation Region [107°W to 83°W, 24°N to 40°N] for which the Events are Rmb Magnitude ≥ 3.0 or Intensity ≥ IV." The updated seismicity within the Gulf of Mexico is listed in FSAR Table 2.5S.2-4, "Seismicity Events Recommended for Recurrence

Analysis within the Gulf of Mexico." SER Figure 2.5S.2-1 depicts the geographic distribution of earthquakes in the applicant's updated earthquake catalog.

**Gulf of Mexico Seismicity – Updates to the EPRI-SOG Catalog**

As shown in SER Figure 2.5S.2-1, the southeastern portion of the 320-km (200-mi) site region extends into the Gulf of Mexico. However, the original EPRI-SOG earthquake catalog covers only a small portion of the Gulf of Mexico along the coastline. The applicant updated the original EPRI-SOG catalog with seismicity within the Gulf of Mexico between latitude 24° N to 32° N and longitude 100° W to 83° W. This update was prompted by the occurrence of two moderate-sized seismic events in the Gulf region. These two events, the M 5.1 event on February 10, 2006, and the M 5.8 event on September 10, 2006, are shown in SER Figure 2.5S.2-1, "Bechtel Group EPRI Source Zones." After updating the earthquake catalog for the Gulf of Mexico region, the applicant developed estimates for completeness periods of earthquakes as a function of magnitude and location. To characterize periods of completeness for the Gulf of Mexico, the applicant divided the seismicity catalog into time frames and the event magnitude scale into intervals. The applicant then determined a probability of completeness for each interval. Using these completeness probabilities and the updated seismicity catalog, the applicant found a slope for the Gutenberg-Richter recurrence relation (i.e., the b value) of 1.055 for the Gulf of Mexico. The applicant asserted that the b value and the maximum likelihood of fit to the data are good. The applicant concluded that the detection probability matrix in FSAR Table 2.5S.2-6 is a reasonable characterization of the completeness of seismicity in the Gulf of Mexico.

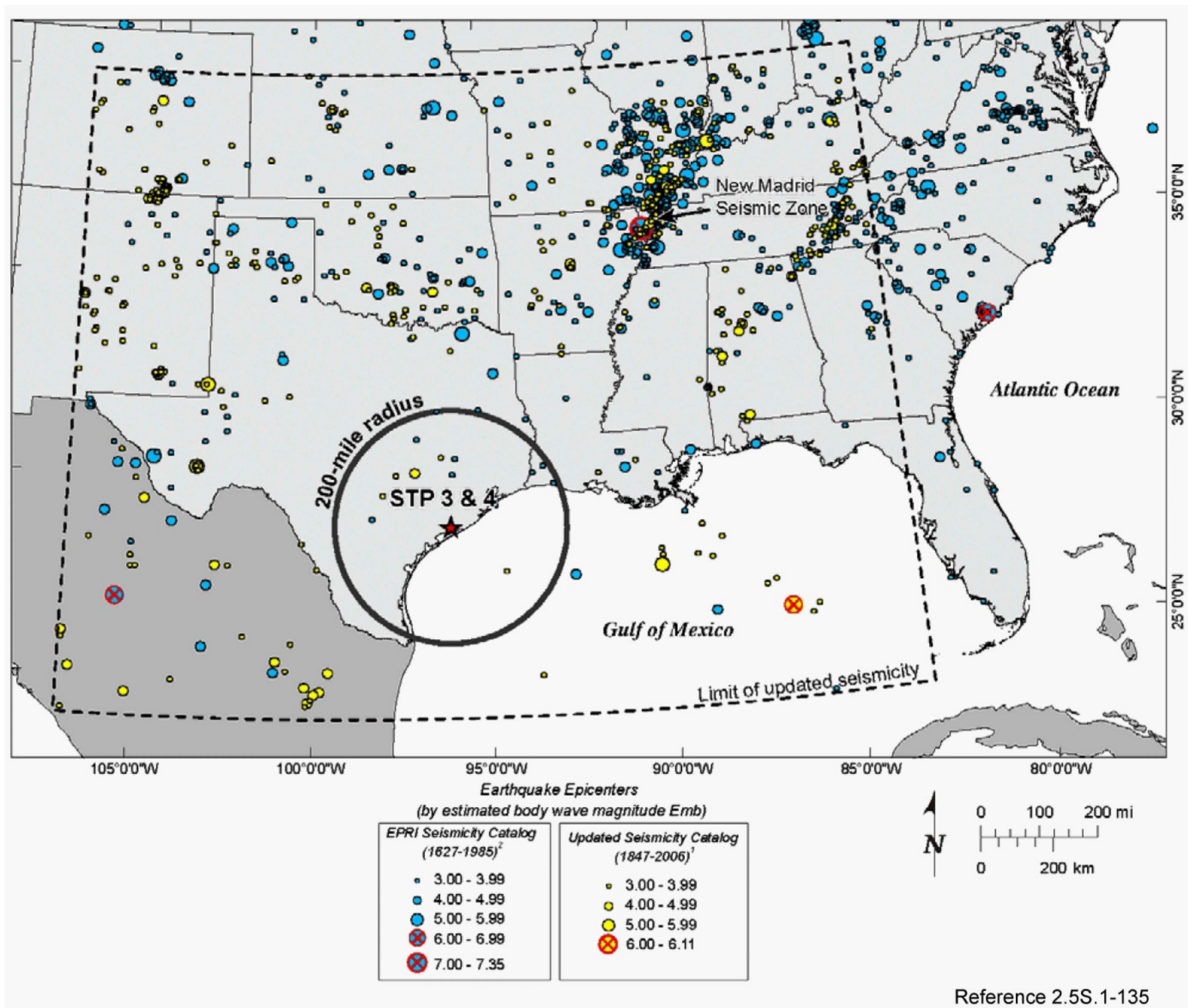


**Figure 2.5S.2-1 Earthquakes (mb > 3) from the EPRI-SOG Seismicity Catalog (Blue Circles) and the Applicant's Updated Seismicity Catalog (Yellow Circles) (FSAR Figure 2.5S.1-26)**

**Mexico and Central America Seismicity**

Additionally, the applicant evaluated the seismicity within the Middle America Trench (MAT), located along the west coast of Mexico and northern Central America, in relation to the potential impact on the seismic hazard at the STP, Units 3 and 4, site. FSAR Subsection 2.5S.2.1.5.1, summarizes the applicant's assessment of the potential impact of major Central American earthquakes associated with the MAT, such as the 1985 magnitude 8.0 earthquake in Mexico, on the seismic hazard at the STP, Units 3 and 4, site. Seismicity within the MAT is located approximately 1,300 km (800 mi) from the STP, Units 3 and 4, site. Later in the FSAR (Subsection 2.5S.2.4.8), the applicant describes the sensitivity study that evaluated the seismic hazard contribution from the MAT, the major source of Central American seismicity. The applicant concluded from the sensitivity study that the MAT's impact on the seismic hazard at the STP, Units 3 and 4, site is not significant. Therefore, seismicity within Mexico and Central America is not considered a major contributor to the seismic hazard at the STP site and is not included in the applicant's updated seismicity catalog.

2.5S.2.2.2 Geologic and Tectonic Characteristics of the Site and Region

FSAR Subsection 2.5S.2.2, describes the original EPRI-SOG (EPRI, 1986) seismic source models that contribute to 99 percent of the total hazard at the STP, Units 1 and 2, site. These contributing EPRI-SOG sources are from the 1986, EPRI-SOG study referenced above. The applicant found that these same seismic sources also contributed to 99 percent of the total hazard at the STP, Units 3 and 4, site. The applicant also reviewed available geological, seismological, and geophysical data from the late 1980s to evaluate the need for modifications to the original seismic source models of the EPRI-SOG ESTs. SER Subsection 2.5S.2.2.4 describes the applicant's sensitivity studies of these potential source zone updates as well as potential new seismic sources.

**Summary of EPRI-SOG Seismic Source Model**

As specified in RG 1.208, the applicant used the 1986, EPRI-SOG seismic source model for the CEUS as a starting point for the seismic source characterization of the STP site. The 1986, EPRI-SOG seismic source model is comprised of input from six independent ESTs that included the Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants. Each team evaluated geological, geophysical, and seismological data to develop a model of seismic sources in the CEUS. The 1989 EPRI-SOG PSHA study (EPRI NP-6395-D, 1989) subsequently incorporated each of the EST models for nuclear power plant sites in the CEUS. FSAR Subsections 2.5S.2.2.1 through 2.5S.2.2.7, describe the primary seismic sources developed by each of the six ESTs that contributed to 99 percent of the total hazard at the STP, Units 3 and 4, site (SER Table 2.5S.2-1).

Bechtel Group. Bechtel Group has two large seismic source zones that contribute to 99 percent of the total hazard at the STP, Units 3 and 4, site: the Gulf Coast Zone (BZ1) and the Texas Platform Zone (BZ2). The Gulf Coast Zone is a background source that encompasses most of the site region, extends from western Texas to eastern Florida, and has an assigned maximum $m_b$ of 6.6. The Texas Platform Zone is an areal source that includes part of the site region, extends from northwestern New Mexico to northern Texas, and has an assigned maximum $m_b$ of 6.6.

Dames & Moore. Dames & Moore have three seismic source zones that contribute to 99 percent of the total hazard at the STP, Units 3 and 4, site: the South Coastal Margin Zone (20), the Ouachitas Fold Belt Zone (25), and the Combination Zone (C08). The South Coastal Margin Zone is a large background source that encompasses most of the site region, extends from Mexico along the Texas coastal plain to eastern Florida, and has an assigned maximum $m_b$ of 7.3. The Ouachitas Fold Belt Zone is located a minimum distance of 171 km (106 mi) from the STP, Units 3 and 4, site, has an assigned maximum $m_b$ of 7.2, encompasses a part of the site region, and characterizes the Oachita mountain belt extending from Arkansas through Oklahoma and the Texas coastal plain into Mexico. The Combination Zone encompasses the Ouachitas Fold Belt Zone (25) while excluding a kink in the Ouachitas fold belt (25A), overlaps part of the STP, Units 3 and 4, site region, and has an assigned maximum $m_b$ of 7.2.

Law Engineering. Law Engineering has two large areal seismic source zones that contribute to 99 percent of the total hazard at the STP, Units 3 and 4, site: the New Mexico Texas Block Zone (124) and the South Coastal Block (126). The New Mexico Texas Block Zone is located 76 mi (122 km) from the STP site; has an assigned maximum $m_b$ of 5.8; and reaches into the site region encompassing most of Texas, the Gulf Coastal Plain, and eastern New Mexico. The South Coastal Bock Zone encompasses most of the site region, extends from Mexico through Texas to eastern Florida, and has an assigned maximum $m_b$ of 4.9.

Rondout Associates. Rondout Associates have one seismic source zone that contributes to 99 percent of the total hazard at the STP, Units 3 and 4, site: the Gulf Coast to Bahamas Fracture Zone (51). The source zone is a large areal source that encompasses most of the site region, extends from Mexico and Texas to eastern Florida, and has a maximum $m_b$ assigned by Rondout Associates of 5.8.

Weston Geophysical Corporation. Western Geophysical Corporation has one seismic source zone that contributes 99 percent of the total hazard at the STP, Units 3 and 4, site: the Gulf Coast Zone (107). This zone is a large areal source that extends from Mexico and Texas to eastern Florida, encompasses most of the site region, and has an assigned maximum $m_b$ of 6.0.

Woodward-Clyde Consultants. Woodward-Clyde Consultants have one seismic source zone contributes 99 percent of the total hazard at the STP, Units 3 and 4, site: the Central United States Backgrounds (B43). This zone is a large areal source centered on the STP, Units 3 and 4, site. The zone is a quadrilateral with sides approximately 6° in length and has an assigned maximum $m_b$ of 6.5.

**Post-EPRI-SOG Seismic Source Characterization Studies**

In accordance with the guidance in RG 1.208, the applicant reviewed seismic source characterization studies published since the original EPRI-SOG (EPRI NP-4726) study. The applicant assessed the need to update the 1986 EPRI-SOG seismic source parameters.

USGS National Seismic Hazard Mapping Project. In FSAR Subsection 2.5S.2.2.8, the applicant stated that since the publication of the 1986, EPRI-SOG study, the USGS National Seismic Hazard Mapping Project (Frankel et al., 2002; 1996) is the one major study that characterized seismic sources within the STP, Units 3 and 4, site region. FSAR Subsection 2.5S.2.2.8, summarizes aspects of this study that are relevant to the STP, Units 3 and 4, site. The summary points out that the 1986 EPRI-SOG CEUS seismic source model incorporates background and local sources each with individual $M_{max}$ distributions, but the USGS source model defines only five distinct source zones for the CEUS with variable $M_{max}$ values.

The STP, Units 3 and 4, site region is primarily encompassed by the USGS Extended Margin Zone, which has an assigned maximum $m_b$ of 7.2. The USGS developed a maximum $m_b$ of 7.2 by comparing the extended margin in the CEUS to analogous tectonic settings worldwide. Because the 1986 EPRI-SOG study previously accounted for relevant hazards around the STP, Units 3 and 4, site, the applicant did not modify hazard calculations or update seismic source models to conform to the 2002 USGS national hazard maps.

2.5S.2.2.3 Correlation of Earthquake Activity with Seismic Sources

FSAR Subsection 2.5S.2.3, describes the correlation between updated seismicity and the EPRI-SOG seismic source models. The applicant compared the distribution of earthquake epicenters from both the original EPRI-SOG historical catalog (1627–1984) and the updated seismicity catalog (1985–2006) with the seismic sources characterized by each EPRI-SOG EST. These comparisons are illustrated in FSAR Figures 2.5S.2-1 through 2.5S.2-6. Based on these comparisons, the applicant concluded that: (1) there are no new earthquakes within the site region that can be associated with a known geologic structure, (2) there are no clusters of seismicity that would suggest a new seismic source not captured by the EPRI-SOG seismic source model, and (3) the updated catalog does not show a pattern of seismicity that would require significant revisions to the geometry of any of the EPRI-SOG seismic sources. However, the earthquakes on September 10, 2006, (M 5.8) and February 10, 2006, (M 5.1) in the Gulf of Mexico prompted the applicant to increase the $M_{max}$ and modify the seismicity parameters (activity rate and b-value) for the Gulf of Mexico seismic source zones defined by the EPRI-SOG ESTs. SER Subsection 2.5S.2.2.4, describes the applicant's updated data in more detail.

2.5S.2.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

In FSAR Subsection 2.5S.2.4, the applicant describes the earthquake potential for the site in terms of the most likely earthquake magnitudes and source-site distances, which are referred to as controlling earthquakes. FSAR Subsection 2.5S.2.4, presents the results of the applicant's PSHA for the STP, Units 3 and 4, site. In performing the PSHA, the applicant followed the guidance in RG 1.208 to determine the seismic hazard curves and controlling earthquakes for the STP site. The seismic hazard curves generated by the applicant's PSHA represent generic hard rock conditions (characterized by a shear [S]-wave velocity of at least 9,200 ft per second [ft/s]). The applicant determined the low- and high-frequency controlling earthquakes by deaggregating the PSHA hazard curves. Deaggregation is the process to determine the controlling earthquake magnitude-distance parameters that dominate the seismic hazard. Before determining the controlling earthquakes, the applicant updated the original 1989 EPRI-SOG PSHA (EPRI NP-6395-D, 1989) using the seismic source zone adjustments and the new ground motion models as described below.

**PSHA Inputs**

Before performing the PSHA, the applicant updated the original 1989 EPRI-SOG PSHA inputs using the updated Gulf of Mexico and Coastal Region seismic sources listed in SER Table 2.5S.2-1, and the NMSZ summarized below. The applicant also performed sensitivity studies for several EPRI-SOG seismic source zones to determine which zones needed to be updated. In addition to these source zone updates, the applicant used the updated 2004 EPRI (EPRI TR-1009684, 2004) ground motion prediction models instead of the 1989 EPRI-SOG (EPRI NP-6395-D) ground motion prediction models that were used in the original 1989 EPRI-SOG PSHA. The applicant also used ground motion prediction uncertainties and weights published by Abrahamson and Bommer (2006) instead of the original uncertainties associated with the 2004 EPRI-SOG (EPRI TR-1009684, 2004) ground motion models.

Seismic Source Models. The applicant updated and performed sensitivity studies for four potentially hazardous seismic sources. The four sources are the Gulf of Mexico and Coastal Region, the NMSZ, the MEEG, and the MAT. The sensitivity analyses revealed that the modifications to EPRI-SOG Gulf Coastal seismic sources and an updated NMSZ (Exelon, 2006) contributed significantly to the STP, Units 3 and 4, site seismic hazard. These updated sources were included in the final PSHA calculation. The applicant found a minimal impact on hazard from the MEEG and MAT. Therefore, the EPRI-SOG seismic sources were not updated with the MEEG source data, and the MAT seismic source was not included in the PSHA calculation. The applicant performed the final STP, Units 3 and 4, PSHA calculations using the updated EPRI-SOG seismic sources listed in SER Table 2.5S.2-1 and Exelon's (2006) updated NMSZ. The applicant characterized seismic sources by reviewing the geological, geophysical, and seismological data used in the 1986 EPRI-SOG study and comparing those data to the data developed since 1986. SER Table 2.5S.2-1 lists the 1986 EPRI-SOG seismic sources that fall within 320 km (200 mi) of the STP, Units 3 and 4, site, which are used as inputs to the applicant's updated PSHA. SER Table 2.5S.2-1 also lists the applicant's updates to the Gulf Coastal sources, as described below. The following SER sections describe the applicant's seismic source updates and sensitivity studies.

**Table 2.5S.2-1 EPRI-SOG EST Seismic Sources that Contribute to 99 Percent of the Total Hazard at STP Units 3 and 4 (FSAR Tables 2.5S.2-7 through 2.5S.2-13)**

| EPRI-SOG EST | Source | Description | Probability Of Activity | $M_{max}$ Distributions EPRI-SOG (1989) $m_b$ [Weights] | Updated $M_{max}$ DISTRIBUTIONS STP 3 And 4 $m_b$ [Weights] |
|---|---|---|---|---|---|
| Bechtel Group | BZ1 | Gulf Coast | 1.0 | 5.4 [0.1]<br>5.7 [0.4]<br>6.0 [0.4]<br>6.6 [0.1] | 6.1 [0.1]<br>6.4 [0.4]<br>6.6 [0.5] |
| | BZ2 | Texas Platform | 0.1 | 5.4 [0.1]<br>5.7 [0.4]<br>6.0 [0.4]<br>6.6 [0.1] | No update |
| Dames & Moore | 20 | South Coastal Margin | 1.0 | 5.3 [0.8]<br>7.3 [0.2] | 5.5 [0.8]<br>7.3 [0.2] |
| | 25 | Ouachitas Fold Belt | 0.35 | 5.5 [0.8]<br>7.2 [0.2] | No update |
| | C08 | Combination | NA | 5.5 [0.8] | No update |

| EPRI-SOG EST | Source | Description | Probability Of Activity | $M_{max}$ Distributions EPRI-SOG (1989) $m_b$ [Weights] | Updated $M_{max}$ DISTRIBUTIONS STP 3 And 4 $m_b$ [Weights] |
|---|---|---|---|---|---|
| | | zone: 25 excluding 25A (Ouachitas Fold Belt excluding Kink in Fold Belt) | | 7.2 [0.2] | |
| Law Engineering | 124 | New Mexico – Texas Block | 1.0 | 4.9 [0.3]<br>5.5 [0.5]<br>5.8 [0.2] | No update |
| | 126 | South Coastal Block | 1.0 | 4.6 [0.9]<br>4.9 [0.1] | 5.5 [0.9]<br>5.7 [0.1] |
| Rondout Associates | 51 | Gulf Coast to Bahamas Fracture Zone | 1.0 | 4.8 [0.2]<br>5.5 [0.6]<br>5.8 [0.2] | 6.1 [0.3]<br>6.3 [0.55]<br>6.5 [0.15] |
| Weston Geophysical Corporation | 107 | Gulf Coast | 1.0 | 5.4 [0.71]<br>6.0 [0.29] | 6.6 [0.89]<br>7.2 [0.11] |
| Woodward-Clyde Consultants | B43 | Central US Backgrounds | NA | 4.9 [0.17]<br>5.4 [0.28]<br>5.8 [0.27]<br>6.5 [0.28] | No update |
| EPRI=Electric Power Research Institute; EST=Earth Science Team; SOG=Seismicity Owner Group; $M_b$=body wave magnitude. | | | | | |

**Gulf of Mexico and Coastal Regions**

Prompted by the occurrence of two moderate-sized earthquakes (M 5.1 event on February 10, 2006, and M 5.8 on September 10, 2006) in the Gulf of Mexico, the applicant updated the seismicity parameter values ($M_{max}$, weight) for the EPRI-SOG EST source zones extending into the Gulf, as described in FSAR Subsection 2.5S.2.4.3. The two moderate-sized events exceed the upper and/or lower bound of the $M_{max}$ distributions used in the original EPRI-SOG Gulf of Mexico and Coastal Region seismic source models. Five seismic sources were updated: Bechtel Group's source BZ1, Dames & Moore's source 20, Law Engineering's source 126, Rondout's source 51, and Weston Geophysical's source 107. The applicant did not update the Woodward-Clyde Consultants' source zone B43, which does extend into the Gulf of Mexico. The applicant stated that source zone B43 (Woodward-Clyde Consultants) is at a sufficient distance from the epicenters of the recent earthquakes it is not updated. SER Table 2.5S.2-1 and FSAR Table 2.5S.2-13, "Comparison of EPRI EST Characterizations of Gulf of Mexico Costal Source Zones and Modifications for STP 3 & 4," list the $M_{max}$ values and associated weights for the applicant's source zones and updates.

**New Madrid Seismic Zone**

The applicant also includes an updated NMSZ source model in the PSHA for the STP, Units 3 and 4, site. The NMSZ, located more than 800 km (500 mi) northeast of the STP site, produced a series of large-magnitude earthquakes in 1811 and 1812. Based on paleoliquifaction research in the epicentral area, researchers have now determined that the 1811-1812 sequence of earthquakes was preceded by repeated earthquakes of a similar size with an approximate 500-year recurrence interval. These large-magnitude events are considered "characteristic earthquakes" for the NMSZ, meaning that the source is capable of producing similar-sized large earthquakes at certain intervals. The updated NMSZ model described in the SSAR for the Clinton ESP site (Exelon, 2006) formed the basis for determining the potential contribution from the NMSZ to the seismic hazard at the STP, Units 3 and 4, site. The Clinton ESP NMSZ model accounts for: (1) new information on recurrence intervals for large earthquakes in the New Madrid area, (2) recent estimates of possible earthquake sizes on each of the active faults, and (3) the possibility of multiple earthquake occurrences within a short period of time (earthquake clusters).

The applicant stated that the following three sources are identified in the NMSZ; each source has two alternative fault geometries that are in parentheses:

- Southern New Madrid (Blytheville Arch/Bootheel Lineament and Blytheville Arch/Blytheville Fault Zone).

- Northern New Madrid (New Madrid North and New Madrid North Plus Extension).

- Reelfoot Fault (Reelfoot Central Section and Reelfoot Full Length).

The applicant calculated the seismic hazard while considering the possibility of clustered earthquake occurrences. The applicant computed the hazard using a simplified model in which all three sources rupture during each "event," which results in a slightly higher ground motion hazard than considering the possibility of two sources rupturing or of a smaller-magnitude earthquake for one of the three ruptures. The applicant developed the occurrence rate of earthquake clusters using a Poisson model and a lognormal renewal model with a range of coefficients of variation (Exelon, 2006). Consistent with Exelon (2006), the applicant assumed that all faults were vertical and extended from the surface to a depth of 20 km (13 mi). A finite rupture model represents extended rupture on all sources. A sensitivity analysis the applicant performed indicated that the updated NMSZ contributed to 99 percent of the hazard at the STP, Units 3 and 4, site. For this reason, the applicant included the NMSZ in the final seismic hazard calculations.

<u>Mt. Enterprise-Elkhart Graben</u>

In FSAR Subsection 2.5S.2.4.4.1, the applicant stated that the MEEG is comprised of a system of east-west striking normal faults located approximately 320 km (200 mi) north-northeast of the STP, Units 3 and 4, site (FSAR Figure 2.5S.1-25). FSAR Subsection 2.5S.1.1.4.4.5.1 describes evidence of possible Quaternary motion along the MEEG, which includes: (1) displaced late Quaternary deposits overlying Eocene strata (Collins et al., 1980), (2) geodetic leveling data indicating relative movement across the center of the MEEG (Collins et al., 1980), and (3) historical and instrumentally located seismicity spatially associated with the MEEG (Frohlich and Davis, 2002). The applicant performed a sensitivity analysis to assess the MEEG source's contribution to the hazard at STP, Units 3 and 4. The applicant's results showed that the updated MEEG source contributed to less than one percent of the hazard. Thus, the applicant excluded the updated MEEG from the final seismic hazard calculations.

Middle America Trench

The MAT is located on the western coast of Mexico more than 1,300 km (800 mi) from the STP, Units 3 and 4, site. However, due to the relatively low levels of seismic activity surrounding the STP, Units 3 and 4, site and the large magnitude events that have occurred along the MAT, the applicant conducted a seismic hazard sensitivity study to assess the potential impact on the STP, Units 3 and 4, site hazard, which is described in FSAR Subsection 2.5S.2.4.8. Due to the large distance of the MAT from the STP, Units 3 and 4, site and the expected crustal attenuation (i.e., the gradual dissipation of seismic energy that occurs while seismic waves travel), the applicant focused the study on 1–hertz (Hz) ground motion attenuation relationships and large-magnitude subduction interplate earthquakes (i.e., the type of earthquake expected along the MAT, which occurs at the boundary between two tectonic plates). The applicant evaluated several different source configurations, as well as seven 1-Hz attenuation relationships for their median attenuation behavior over the magnitude range of 6.5 to 8.5 and for distances up to 2,000 km (1240 mi). The applicant compared the 1-Hz hazard curve that resulted from the PSHA (including the MAT seismic source) to the 1-Hz curve from a PSHA that included only the significant updates to the EPRI-SOG sources. For both the $10^{-4}$ and $10^{-5}$ hazard levels of the total hazard curve, the applicant found that the MAT seismic source contributed to less than 1 percent of the total hazard. For this reason, the applicant did not include the MAT seismic source in the final seismic hazard calculations for the STP, Units 3 and 4, site.

Ground Motion Models. The applicant used the ground motion models developed by the 2004 EPRI-sponsored study (EPRI TR-1009684, 2004) for the updated PSHA. The 2004 EPRI project reviewed the latest knowledge of CEUS ground motions. The study updated equations estimating median spectral acceleration and associated uncertainties as a function of earthquake magnitude and distance throughout the CEUS. Epistemic uncertainty, which results from limits of knowledge, was modeled using multiple ground motion equations with weights and multiple estimates of weighted aleatory uncertainty reflecting the inherent randomness in the data. The aleatory uncertainties were later reexamined by EPRI (EPRI 2006) resulting in modified aleatory uncertainties and weights. The 2006 EPRI study found that the aleatory uncertainties were too large in 2004 EPRI report, thus resulting in an overestimation of the seismic hazard. Therefore, the applicant used the 2004 EPRI ground motion models with the updated 2006 EPRI aleatory uncertainty equations.

**PSHA Methodology and Calculation**

Using the updated EPRI-SOG seismic source characteristics and new ground motion models with updated uncertainties as inputs, the applicant performed PSHA calculations for peak ground acceleration (PGA) and spectral acceleration at frequencies of 25, 10, 5, 2.5, 1, and 0.5 Hz. Following the guidance in RG 1.208, the applicant performed PSHA calculations that assumed generic hard rock site conditions at the STP, Units 3 and 4, site (i.e., an S-wave velocity of at least 2.8 km [9,200 ft/s]).

**PSHA Results**

The applicant performed the STP, Units 3 and 4, PSHA calculations using the EPRI-SOG seismic sources listed in SER Table 2.5S.2-1 and Exelon's updated NMSZ (Exelon, 2006). Site seismic hazard characteristics are quantified by the seismic hazard curves from the PSHA and the uniform hazard response spectra (UHRS) that cover a broad range of natural frequencies. The hazard curves were developed identifying and characterizing each seismic source that contributed to 99 percent of the seismic hazard at the STP, Units 3 and 4, site, while the UHRS is a plot of spectral acceleration that has an equal likelihood of exceedance at different frequencies. FSAR Figures 2.5S.2-18 through 2.5S.2-24, illustrate the applicant's mean and the 5th, 16th, 50th, 84th, and 95th fractile hard rock hazard curves for the PGA and spectral acceleration at frequencies of 25, 10, 5, 2.5, 1, and 0.5 Hz. SER Figure 2.5S.2-2 shows the mean and median UHRS for the $10^{-4,}$ $10^{-5}$, and $10^{-6}$ annual frequencies of exceedance for hard rock conditions, which the applicant generated from the seismic hazard curves in FSAR Figures 2.5S.2-18 through 2.5S.2-24. The mean UHRS values are also in FSAR Table 2.5S.2-16, "Mean Rock Uniform Hazard Response Spectral Accelerations (g)."

The applicant then described the earthquake potential for the site in terms of the most likely earthquake magnitudes and source-to-site distances, which are referred to as controlling earthquakes. The applicant determined the controlling earthquakes that dominate low frequencies (LF) and the high frequencies (HF). To determine these controlling earthquakes, the applicant performed deaggregation of the PSHA at selected probability levels. The procedure the applicant used is outlined in RG 1.208. The applicant chose to perform the deaggregation of the mean $10^{-4}$, $10^{-5}$, and $10^{-6}$ PSHA hazard results. The applicant's complete deaggregation results are in FSAR Figures 2.5S.2-27 through 2.5S.2-32. The applicant noted that the last distance interval shown on these plots represents source contributions from a distance of 400 km (248 mi) or greater.

Based on the deaggregation plots in FSAR Figures 2.5S.2-27 through 2.5S.2-32, the applicant concluded that for the $10^{-4}$ and $10^{-5}$ annual frequency of exceedance, the NMSZ is the largest contributor to the seismic hazard for both high and low frequencies. The applicant stated that for the $10^{-5}$ annual frequency of exceedance (FSAR Figures 2.5S.2-29 and 2.5S.2-30), the contribution of the NMSZ is smaller, particularly for high frequencies where the hazard contribution from local sources is also significant. The applicant also noted that for an annual frequency of exceedance of $10^{-6}$, virtually all of the hazard at high frequencies comes from local sources, while low frequencies have about equal contributions from the NMSZ and from local sources.

SER Table 2.5S.2-2, "Seismicity Catalog for pre-1985 for the Gulf of Mexico," includes the mean magnitudes and distances resulting from the applicant's hazard deaggregations. Following the guidance of RG 1.208, the applicant selected the controlling earthquake for the low-frequency ground motions from the R > 100 km (63 mi) calculation (R is source-to-site distance); the controlling earthquake for the high-frequency ground motions is from the overall calculation. The resulting controlling earthquakes are depicted by the shaded cells in SER Table 2.5S.2-2.

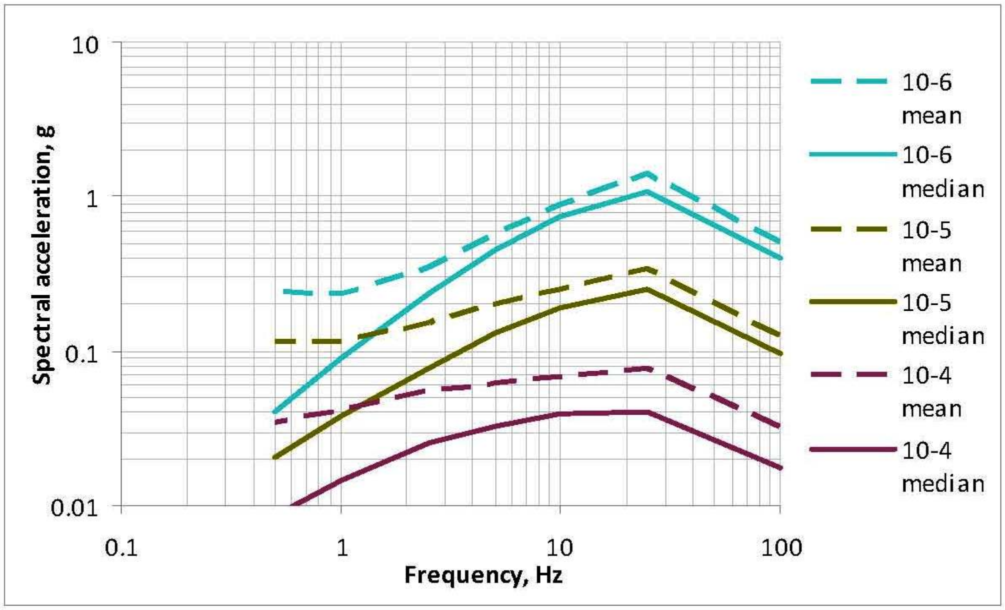


**Figure 2.5S.2-2 Mean and Median Rock Uniform Hazard Response Spectra (UHRS) (FSAR Figure 2.5S.2-26)**

**Table 2.5S.2-2 Controlling Earthquakes for Different Annual Frequencies of Exceedance and Structural Frequencies (FSAR Table 2.5S.2-17)**

| STRUCTURAL FREQUENCY (Hz) | ANNUAL FREQUENCY OF EXCEEDANCE | OVERALL HAZARD | | HAZARD FROM R > 100 km | |
|---|---|---|---|---|---|
| | | M | R (km) | M | R (km) |
| 1 & 2.5 | $10^{-4}$ | 7.4 | 600 | 7.6 | 880 |
| 5 & 10 | $10^{-4}$ | 6.7 | 230 | 7.5 | 790 |
| 1 & 2.5 | $10^{-5}$ | 7.3 | 380 | 7.7 | 890 |
| 5 & 10 | $10^{-5}$ | 6.1 | 46 | 7.7 | 850 |
| 1 & 2.5 | $10^{-6}$ | 6.9 | 112 | 7.8 | 890 |
| 5 & 10 | $10^{-6}$ | 5.6 | 10 | 7.8 | 860 |

M=magnitude, R=source-to-site distance, Hz=frequency in cycle per second, and km=kilometer.

The applicant's representative controlling earthquakes are shaded in gray.

The applicant then developed the smooth rock UHRS from the mean UHRS amplitudes, as shown in FSAR Table 2.5S.2-216 (and SER Figure 2.5S.2-2), using the controlling earthquake magnitude and distance values in SER Table 2.5S.2-2 and the hard rock spectral shapes for

CEUS earthquake ground motions recommended in NUREG/CR–6728, "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-Consistent Ground Motion Spectra Guidelines." The resulting $10^{-4}$ and $10^{-5}$ smoothed spectra are shown in SER Figure 2.5S.2-3.

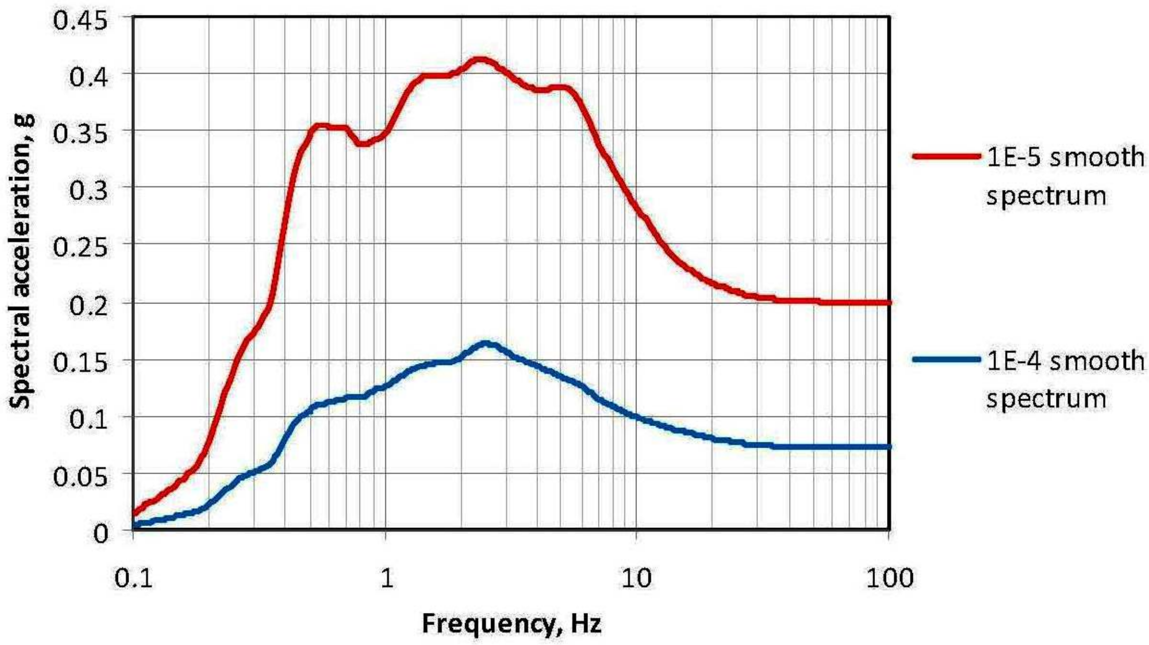


**Figure 2.5S.2-3 Smooth $10^{-4}$ and $10^{-5}$ rock UHRS (FSAR Figure 2.5S.2-51)**

2.5S.2.2.5 Seismic Wave Transmission Characteristics of the Site

FSAR Subsection 2.5S.2.5, describes the procedure used by the applicant to develop the amplification or deamplification effects of soils on seismic wave transmission beneath the site. The hazard curves generated by the PSHA are defined for generic hard rock conditions (characterized by an S-wave velocity of 9,200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 9,144 m (30,000 ft) beneath the ground surface at the STP, Units 3 and 4, site. The applicant modeled the effects of the overlying soil by using a truncated soil column, which extends to a depth of 2,469 m (8,100 ft) below the ground surface. To determine the soil UHRS, the applicant: (1) developed soil models for the STP, Units 3 and 4, site; (2) randomized the soil profiles to account for variability; and (3) performed the final site response analysis.

**Site Response Model**

The applicant stated that the subsurface geology at the STP, Units 3 and 4, site consists of deep marine and fluvial deposits overlying bedrock. Based on the results of test borings, CPT, test pits, and geophysical testing, the applicant divided the upper 182 m (600 ft) of the site's soil profile into 12 units, which mainly consist of alternating layers of very stiff to hard silty clay and dense to very dense silty sand. To determine and estimate the soil's various seismic wave propagation properties such as seismic wave velocity, Possion's ratio, and shear modulus, the applicant used P-S Suspension Logging measurements and Seismic Cone Penetration Testing results to a depth of 182 m (600 ft). The applicant estimated another parameter, kappa (κ), as input into the site response analysis. Kappa is the near-surface damping parameter, which is an estimate of the dissipation of seismic energy of the site during an earthquake due to damping within soil layers and waveform scattering at layer boundaries. The applicant adopted a base case κ value of 0.040 s with a standard deviation of 0.4 natural log units based on the EPRI research (EPRI, 1993; 2005). Layer damping is an assumed additive for soil layers and is dependent on the individual soil layers. The applicant used site-specific geophysical data (S-wave velocities [$V_S$]), the generic EPRI shear modulus (EPRI, 1993), and damping ratio curves to determine a value for κ for each soil layer above a depth of 182 m (600 ft). The applicant then subtracted the κ value calculated for the upper 182 m (600 ft) from the total or base case κ value (0.040 s) to obtain a constant damping value for the soil layers below 182 m (600 ft). As described in detail in FSAR Section 2.5S.4, the applicant then used these data to develop a base case profile for the upper 182 m (600 ft) of soil. Below a depth of 182 m (600 ft), the applicant used sonic log data to determine the soil's seismic properties. The applicant truncated the soil profile model at a depth of 2,469 m (8,100 ft), because this depth captures site response in the range of frequencies of interest—greater than 0.1 Hz. FSAR Figure 2.5S.2-35a, "Best Estimate Soil Column Frequency," illustrates that at depths greater than 2,160 m (7,000 ft), the soil column frequency is less than 0.1 Hz.

Using the model of Silva et al. (Silva et al., 1996), the base case seismic wave velocity profiles, and associated shear moduli and damping parameters, the applicant generated 60 artificial randomized soil seismic wave property profiles in order to account for variations in soil properties across the site. The applicant's resulting randomized seismic wave velocity profiles are depicted in FSAR Figure 2.5S.2-36, while the randomized shear modulus degradation and damping ratio curves for one of the soil layers are depicted in FSAR Figures 2.5S.2-37, "Strain Dependent Degradation Curves for Stratum C," and 2.5S.2-38, "Strain Dependent Damping Ratio Properties for Stratum C," respectively. The applicant used these randomized profiles as input to the site response calculations that are summarized below.

**Site Response Methodology and Results**

The applicant used Random Vibration Theory (RVT) to calculate site response at STP, Units 3 and 4. Most site response analysis studies are performed using the approach used in the well-known computer program SHAKE (Idriss and Sun, 1992; Schnabel et al., 1972). To minimize soil nonlinearity effects, applicants using the SHAKE program to calculate site response utilize 60 individual acceleration time histories as design input motions into the site-response analysis. RVT, however, eliminates the need for generating multiple acceleration time histories by utilizing input response spectra as design input motions into the site-response analysis. Response spectra do not illustrate acceleration through time as acceleration time histories do; response spectra show the strength of the seismic energy as a function of frequency. RVT is an NRC-accepted method for estimating site response, as described in RG 1.208. The RVT method requires: (1) input of the hard rock UHRS as the input response spectra, (2) the 60 randomized soil seismic wave property profiles, and (3) an effective strain ratio. The outputs of an RVT analysis are response spectra defined at the ground surface (i.e., a ground surface UHRS), which accounts for the effects of soil amplification (or deamplification) on the hard rock UHRS. The applicant calculated the strong-motion duration using mean magnitudes and distances from the STP, Units 3 and 4, controlling earthquakes, as well as values of crustal seismic wave velocity and stress drop that are typical for eastern North America. The applicant used a value of 0.65 for the effective strain ratio. To calculate the site amplification effects of the soil, the applicant divided the ground surface UHRS by the hard rock UHRS. This division results in 4 mean amplification functions by combining the results of low and high frequencies and $10^{-4}$ and

$10^{-5}$ input spectra. The applicant's resulting amplification functions are depicted in FSAR Figure 2.5S.2-49a, "Comparison of Log-Mean Soil Transfer Functions (Amplification Factors) at the Ground Surface Level for LF and HF 10-4 and 10-5 Input Motions." According to the applicant's results in FSAR Figure 2.5S.2-49a, the STP, Units 3 and 4, site subsurface amplifies the $10^{-4}$ LF, $10^{-4}$ HF, $10^{-5}$ LF, and $10^{-5}$ HF input hard rock motion over the fairly wide frequency range of 0.1 to ~10 Hz and from ~ 60 to 100 Hz, with the maximum amplification of ~4.0 at a frequency of 0.25 Hz. Deamplification occurs between ~10 and 60 Hz. Lastly, the applicant developed envelope spectra for the $10^{-4}$ and $10^{-5}$ ground surface UHRS, which combine the individual low- and high frequency results. The applicant smoothed these spectra using a running average filter (shown in SER Figure 2.5S.2-4).

2.5S.2.2.6 Ground Motion Response Spectra

FSAR Subsection 2.5S.2.6, describes the method the applicant used to develop the horizontal and vertical site-specific GMRS. To calculate the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and in the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) Standard 43-05. The applicant developed the vertical GMRS by developing vertical-to-horizontal response spectral (V/H) ratios based on NUREG/CR–6728, before using the performance-based approach. The applicant followed the procedure referred to as Approach 2A in NUREG/CR–6769, "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Development of Hazard- and Risk-Consistent Seismic Spectra for Two Sites." The applicant's procedure used the smoothed $10^{-4}$ and $10^{-5}$ ground surface UHRS to develop the horizontal and vertical GMRS shown in SER Figures 2.5S.2-4 and 2.5S.2-5, respectively.

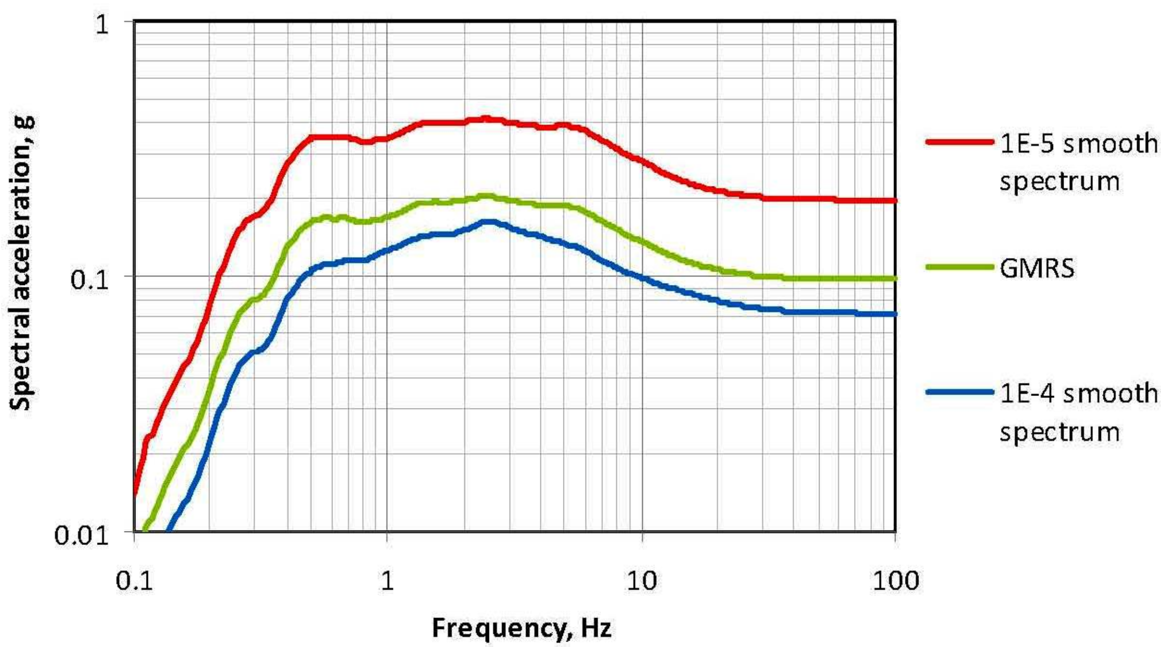


**Figure 2.5S.2-4 Smoothed $10^{-4}$ and $10^{-5}$ Ground Surface (Soil) Horizontal UHRS and Resulting Horizontal GMRS (FSAR Figure 2.5S.2-52)**

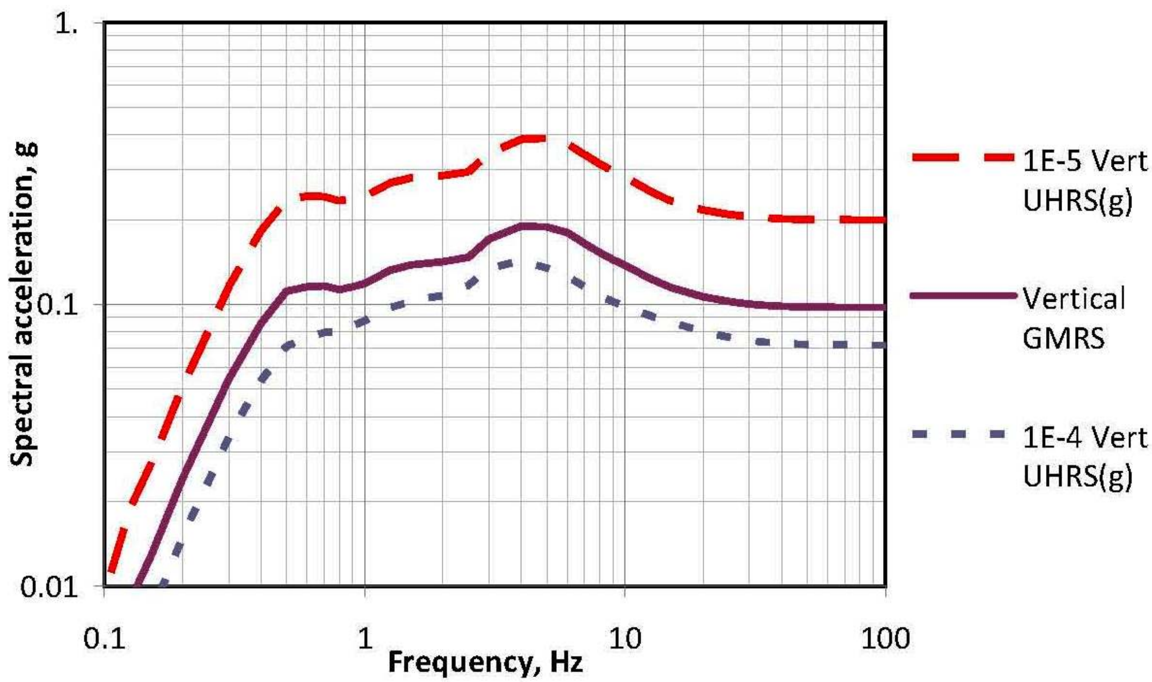


**Figure 2.5S.2-5 Vertical $10^{-4}$ and $10^{-5}$ Soil UHRS and Resulting Vertical GMRS (Referred to in the figure title as DRS in FSAR Figure 2.5S.2-54)**

**Horizontal GMRS**

The applicant developed a horizontal site-specific, performance-based GMRS using the method described in RG 1.208 and in ASCE/SEI Standard 43-05. This performance-based method achieves the annual target performance goal ($P_F$) of $10^{-5}$ per year for frequency-of-onset of significant inelastic deformation. The horizontal GMRS (for each spectral frequency), which meets the $P_F$, is obtained by scaling the smoothed soil $10^{-4}$ UHRS by the design factor:

$$DF = 0.6(A_R)^{0.8} \qquad \text{Equation 2.5.2-3}$$

In SER Equation 2.5.2-3, $A_R$ is the ratio of the $10^{-5}$ UHRS and the $10^{-4}$ UHRS spectral accelerations for each spectral frequency. The applicant's resulting horizontal GMRS (SER Figure 2.5S.2-4) is defined at the top of ground surface.

**Vertical GMRS**

Within FSAR Subsection 2.5S.2.6, the applicant describes the methodology used to calculate the vertical GMRS curve. The applicant obtained the CEUS V/H spectral ratios from NUREG/CR–6728 and multiplied the horizontal UHRS with these ratios to obtain the vertical UHRS at the site. Then, using the same performance-based methodology described in RG 1.208, the applicant calculated the vertical GMRS. Ultimately, the applicant used V/H values from RG 1.60 Revision 1, "Design Response Spectra for Seismic Design of Nuclear Power Plants," because they are conservative and simple when comparing the values obtained from other methods. SER Figure 2.5S.2-5 shows the vertical GMRS at the STP, Units 3 and 4, site, as well as the vertical ground surface (soil) UHRS for both the $10^{-4}$ and $10^{-5}$ mean hazard levels.

### *2.5S.2.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the vibratory ground motion, and the associated acceptance criteria, are in Section 2.5.2 of NUREG–0800. The acceptance criteria for reviewing COL License Information Item 2.24 are in Section 2.5.2 of NUREG–0800.

In particular, the applicable regulatory requirements for reviewing the applicant's discussion of vibratory ground motion are the following:

- 10 CFR 100.23, "Geologic and seismic siting criteria," with respect to obtaining geologic and seismic information necessary to determine site suitability and ascertain that any new information derived from site-specific investigations does not impact the GMRS derived from a probabilistic seismic hazard analysis. In complying with this regulation, the applicant also meets the guidance in RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," and RG 1.208.

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

The following related acceptance criteria are summarized from SRP Section 2.5.2:

- <u>Seismicity</u>: To meet the requirements in 10 CFR 100.23, this subsection is accepted when the complete historical record of earthquakes in the region is listed and when all available parameters are given for each earthquake in the historical record.

- <u>Geologic and Tectonic Characteristics of Site and Region</u>: Seismic sources identified and characterized by the Lawrence Livermore National Laboratory (LLNL) and the EPRI were used for studies in the CEUS in the past.

- <u>Correlation of Earthquake Activity with Seismic Sources</u>: To meet the requirements in 10 CFR 100.23, acceptance of this subsection is based on the development of the relationship between the history of earthquake activity and seismic sources of a region.

- <u>Probabilistic Seismic Hazard Analysis and Controlling Earthquakes</u>: For CEUS sites relying on LLNL or EPRI-SOG methods and databases, the staff will review the applicant's PSHA, including the underlying assumptions and how the results of the site investigations are used to update the existing sources in the PSHA, how they are used to develop additional sources, or how they are used to develop a new database.

- <u>Seismic Wave Transmission Characteristics of the Site</u>: In the PSHA procedure described in RG 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," the controlling earthquakes are determined for generic rock conditions.

- <u>Ground Motion Response Spectra</u>: In this subsection, the staff reviews the applicant's procedure to determine the GMRS.

In addition, the seismic characteristics should be consistent with appropriate sections from RG 1.132, RG 1.206, and RG 1.208.

### *2.5S.2.4 Technical Evaluation*

The staff reviewed the information in Section 2.5S.2 of the STP, Units 3 and 4, COL FSAR. In this section, the applicant provides supplemental information to address COL License Information Item 2.24.

*<u>COL License Information Items</u>*

- COL License Information Item 2.24 Vibratory Ground Motion

The staff reviewed the FSAR for STP, Units 3 and 4, related to COL License Information Item 2.24, which addresses the provision for site-specific information related to the vibratory ground motion aspects of the site including seismicity, geologic and tectonic characteristics, the correlation between earthquake activity and seismic sources, a probabilistic seismic hazard analysis, seismic wave transmission characteristics, and the SSE ground motion.

SER Subsection 2.5S.2.4, includes the staff’s evaluation of the seismic, geologic, geophysical, and geotechnical investigations carried out by the applicant to determine the site-specific GMRS or the SSE ground motion for the site.

The development of the GMRS is based on a detailed evaluation of earthquake potential that takes into account the regional and local geology, Quaternary tectonics, seismicity, and site specific geotechnical engineering characteristics of the site subsurface material.

During the early site investigation stage, the staff visited the site and interacted with the applicant regarding the geologic, seismic, and geotechnical investigations conducted for the STP, Units 3 and 4, COL application. To thoroughly evaluate the geologic, seismic and geophysical information the applicant collected, the staff obtained additional assistance from experts at the USGS. With the USGS advisors, the staff made an additional visit to the STP, Units 3 and 4, site in August 2008, to confirm the applicant’s interpretations, assumptions, and conclusions related to potential geologic and seismic hazards and included in COL FSAR Section 2.5S.2. The staff’s evaluation of this information and of the applicant’s responses to RAIs is presented below.

#### 2.5S.2.4.1 Seismicity

To characterize the seismic hazard for the STP, Units 3 and 4, site, the applicant followed the methodology in RG 1.208 and used the EPRI-SOG seismic hazard models developed in the late1980s (EPRI, 1986) as a starting point. The EPRI-SOG study used an earthquake catalog compiled through 1984 that covers the CEUS. FSAR Subsection 2.5S.2.1 describes the applicant’s update of the original EPRI-SOG earthquake catalog that extended it from 1985 through November 2006. The update also expanded the coverage to include the portions of the Gulf of Mexico that were not covered in the original EPRI-SOG catalog. The applicant also evaluated the seismicity along the west coast of Mexico and northern Central America to determine the potential impact on the seismic hazard for the STP, Units 3 and 4, site.

**EPRI-SOG Seismicity Catalog Updates**

To update the EPRI-SOG earthquake catalog for the region surrounding the STP, Units 3 and 4 site, the applicant evaluated several different earthquake catalogs including those from the ANSS, ISC, and PDE. For each catalog, the applicant compiled the events that had occurred in 1985 and later (through November 2006) in the latitude-longitude window of 24° to 40° N and 107° to 83° W. After eliminating duplicate events, the applicant converted the different magnitude scales used by these catalogs to $m_b$, which is the magnitude scale used in the original EPRI-SOG earthquake catalog. Once the applicant added these more recent events (1985 through 2006) to the original EPRI-SOG catalog, the applicant converted all of the events in this updated catalog to the now commonly used M.

In FSAR Subsection 2.5S.2.1.2, the applicant described the conversion of the different magnitude scales from each of the catalogs to $m_b$ and the subsequent conversion to M. The staff issued RAI 02.05.02-1 and RAI 02.05.02-2, requesting the applicant to clarify two of the steps in this process. In RAI 02.05.02-1, the staff asked how the applicant had determined the uncertainty in the conversion to $m_b$. In RAI 02.05.02-2, the staff asked for the specific conversion equation the applicant had used. In its response to RAI 02.05.02-1 dated July 9, 2008 (ML081960070), the applicant included a table showing how the uncertainty in the magnitude estimates varies with each of the different magnitudes. In its response to RAI 02.05.02-2, dated July 9, 2008, the applicant clarified the conversion equation. As a result of these two RAI responses, the staff was able to follow each step in the magnitude conversion process and to verify that the applicant had used an established procedure to adequately estimate the magnitudes of the earthquakes in the updated earthquake catalog for the site. Therefore, RAI 02.05.02-1 and RAI 02.05.02-2, are resolved and closed.

**Gulf of Mexico Seismicity**

The EPRI-SOG earthquake catalog did not include events from the Gulf of Mexico except along its immediate coastline. The applicant therefore conducted an extensive study in order to comprehensively cover the seismicity in the Gulf of Mexico between latitudes 24° and 32° N and between longitudes 100° and 83° W. The applicant's update was prompted in large part by two recent moderate seismic events in the Gulf, namely an M 5.1 event that occurred on February 10, 2006, offshore of the Louisiana coast; and an M 5.8 event that occurred on September 10, 2006, offshore of the Florida coast. To develop a Gulf of Mexico seismicity catalog for the STP site, the applicant examined 10 different earthquake catalogs. After eliminating duplicate as well as dependent events (foreshocks and aftershocks), the applicant converted each of the different magnitude scales to $m_b$.

The staff issued RAI 02.05.02-3, requesting the applicant to clarify the equation used to convert surface wave magnitudes ($M_s$) to $m_b$ for the Gulf earthquakes. In its response to RAI 02.05.02-3, dated July 9, 2008, the applicant provided the conversion equation and also updates FSAR Subsection 2.5S.2.1.3, to clarify the use of this conversion equation. Based on the applicant's response, the staff was able to verify that the applicant had used an established magnitude conversion. The applicant had thus adequately converted the Gulf earthquakes with the $M_s$ scale to the $m_b$ scale. Therefore, RAI 02.05.02-3 is resolved and closed.

The staff issued RAI 02.05.02-4, requesting the applicant to explain the use of the terms "MAIN vs. non-MAIN," in the context of removing dependent earthquakes (foreshocks and aftershocks) from the Gulf of Mexico seismicity catalog. In its response to RAI 02.05.02-4, dated July 9, 2008, the applicant stated that “MAIN” refers to independent events and “non-MAIN” refers to dependent events. The applicant then explained the method used to merge the Gulf of Mexico events in the original EPRI-SOG catalog with the Gulf earthquakes from the 10 other earthquake catalogs. As a result of the applicant’s response, the staff was able to follow the steps the applicant had used to add Gulf Coast earthquakes to the updated seismicity catalog for the STP, Units 3 and 4, site. The staff verified that the applicant had adequately characterized the seismicity in the Gulf of Mexico. Therefore, RAI 02.05.02-4 is resolved and closed.

To develop recurrence parameters for the Gulf of Mexico earthquakes, the applicant used the previously approved EPRI-SOG methodology. Specifically, the applicant estimated probabilities of earthquake detection for the Gulf of Mexico. FSAR Table 2.5S.2-6, shows that detection probabilities vary from 0.00 (for the magnitude intervals of 3.3 to 3.9 ($m_b$) during the years 1625 to 1779) to 0.30 for the same magnitude range during the years 1980 to 2006. In general, detection probabilities are lower for early time periods and for smaller magnitude earthquakes. Using these detection probabilities for the Gulf of Mexico as well as the Gulf seismicity catalog, the applicant found that the slope of the commonly used Gutenberg-Richter recurrence relation (the b value) is about 1.055. This b value agrees strongly with the b values of other CEUS regions, which are about 1.0.

The staff issued RAI 02.05.02-6, requesting the applicant for additional details clarifying the basis for the assumption that detection probabilities for Gulf of Mexico earthquakes increase with time and with larger magnitudes. In its response to RAI 02.05.02-6, dated July 9, 2008, the applicant explained the development of the matrix of detection probabilities for the Gulf of Mexico to cover the time period from 1625, to the present. The applicant also stated that the two major factors affecting earthquake detection are a population available to feel the earthquakes and the distribution of seismic instruments to record the earthquakes. Over time, both populations and seismic instrumentation have generally increased, and therefore, the detection capability has improved with time. The staff concurred with the applicant that the area of seismic detection capability generally improves with increases in population and seismic instruments. Therefore, RAI 02.05.02-6 is resolved and closed.

The staff issued RAI 02.05.02-7, requesting the applicant to further clarify the determination of 1.055 as the b value for the Gulf of Mexico. In its response to RAI 02.05.02-7, dated July 9, 2008, the applicant stated that the preliminary seismicity analysis for the Gulf of Mexico found a b value of about 0.5, which is well below the typical b value of about 1.0. After modifying the probability of detection values for larger earthquakes ($m_b$ 5.7 to 6.29 and 6.3 to 7.5) during earlier time intervals (1900 to 1924 and 1925 to 1949), the applicant computed a b value of 1.055. The staff examined the modified probabilities in FSAR Table 2.5S.2-6, “Matrix of Detection Probability for the Gulf of Mexico,” and found them to be reasonable after considering the magnitude ranges and time intervals. In addition, the staff concurred with the applicant’s reasoning that a b value of 0.5 is much too low and unlikely for the Gulf of Mexico. The staff therefore concluded that the applicant has adequately characterized the recurrence values for the Gulf of Mexico earthquakes. Therefore, RAI 02.05.02 -7 is resolved and closed.

**Staff Conclusions Regarding Seismicity**

The staff reviewed FSAR Subsection 2.5S.2.1, and concluded that the applicant has developed a complete and accurate earthquake catalog for the region surrounding the STP site, including the Gulf of Mexico seismicity, detection probabilities, and recurrence values. The staff concluded that the seismicity catalog described by the applicant in FSAR Subsection 2.5S.2.1 forms an adequate basis for the seismic hazard characterization of the site and meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

2.5S.2.4.2 Geologic and Tectonic Characteristics of the Site and Region

This section of the safety evaluation includes the staff's evaluation of the seismic source models the applicant uses as part of the PSHA for the STP site. The applicant described seismic source models in FSAR Sections 2.5S.2.2 and 2.5S.2.4. FSAR Subsection 2.5S.2.2, describes the significant seismic sources from the original EPRI-SOG seismic source models (EPRI NP-4726) that contribute to 99 percent of the total hazard at the STP, Units 1 and 2, site. These seismic source models were developed in 1986 by the six EPRI-SOG ESTs. FSAR Subsection 2.5S.2.4 describes the applicant's sensitivity studies that were used to determine whether the 1986 EPRI-SOG seismic source models needed to be updated based on more recent studies in the geologic and seismic literature. As specified in RG 1.208, the applicant evaluated more recent seismic hazard studies and data available for the region surrounding the site, which the applicant compared to the 1986 EPRI-SOG seismic source models. As a result of this evaluation, the applicant updated several of the original source models developed by the six EPRI-SOG ESTs.

**Original EPRI-SOG Seismic Sources**

The six ESTs involved in the EPRI-SOG project were: (1) the Bechtel Group, (2) Dames and Moore, (3) Law Engineering, (4) Rondout Associates, (5) Weston Geophysical Corporation, and (6) Woodward-Clyde Consultants. The ESTs each produced a report with detailed descriptions of their individual philosophy and the methodology they used to identify tectonic features, to evaluate tectonic features as seismic sources, and to develop parameters for the seismic sources. In FSAR Subsection 2.5S.2.2, the applicant briefly describes each seismic source model the six ESTs used that contributes to 99 percent of the total hazard at the STP, Units 1 and 2, site. The ESTs based this determination on the 1986 EPRI-SOG Project. Key parameters that the ESTs used to model the seismic sources include the: (1) source geometries or configurations, (2) $M_{max}$ range and distribution, (3) activity probabilities (Pa), and (4) recurrence values within the seismic source. For the most part, rather than attempting to characterize the seismic potential of known faults or other features in the CEUS, the EPRI-SOG ESTs used areal source zones that encompass many of these structural features. Other source zones encompass areas of focused seismicity or evidence of prehistoric seismic activity, such as paleoliquefaction features.

**Post-EPRI-SOG Seismic Source Studies**

Since the development of the 1986 EPRI-SOG study, only the USGS National Seismic Hazard Mapping Project has characterized the seismic sources within the STP, Units 3 and 4, site region. FSAR Subsection 2.5S.2.2.8, briefly describes the 2002 USGS Hazard map (Frankel et al., 2002) and the similarities and differences between the 1986 EPRI-SOG seismic source models and the 2002 USGS sources. The applicant finds the main difference to be the development by the EPRI-SOG ESTs of many seismic sources, each with individual source geometries and parameters such as $M_{max}$ and recurrences. In contrast, the USGS model for the CEUS defines considerably fewer distinct source zones. In particular, the majority of the region surrounding the STP site is modeled by just one USGS source zone, which is referred to as the

USGS extended margin. The USGS assigned an $M_{max}$ value of 7.5 (**M**) to its extended margin source zone based on analogous extended margins of SCRs worldwide.

The staff issued RAI 02.05.02-10, requesting the applicant to elaborate on the comparison between the USGS seismic source modeling approach and the approach taken by the ESTs for the EPRI-SOG seismic source characterization. Specifically, the staff asked the applicant to justify the claim that the USGS uses only a "small number of sources." In its response to RAI 02.05.02-10, dated September 4, 2008 (ML082530449), the applicant updated this comparison of the USGS and the EPRI-SOG EST modeling approaches in FSAR Subsection 2.5S.2.2.8. The applicant also provided a more refined description of the USGS seismic Extended Source Zone model. The staff reviewed the updated FSAR subsections and concluded that the applicant has adequately described the differences between the USGS and the EPRI-SOG EST approaches. The applicant also adequately described the USGS model of the CEUS extended margin, including the basis for the USGS $M_{max}$ value of 7.5 (**M**) for this source zone. Therefore, RAI 02.05.02-10 is resolved and closed.

The staff issued RAI 02.05.02-5, asking whether the applicant had considered the more recent studies by Johnston et al. (1994) on worldwide earthquakes in SCRs as potential sources for updating the EPRI-SOG seismic source models. In its response to RAI 02.05.02-5, dated September 4, 2008, the applicant stated that earlier versions of the Johnston and Kanter studies were available to the EPRI-SOG ESTs as they developed their source models for the CEUS. As a result, the applicant concluded that these assessments do not constitute new information, thus requiring no update of the EPRI-SOG source characterizations. Based on the availability to the EPRI-SOG ESTs of earlier versions of these two studies, the staff concluded that the EPRI-SOG seismic source models adequately considered worldwide earthquakes in SCRs. The staff concurred with the applicant that the main findings of the Johnston et.al (1994) studies do not constitute information that was not available to the EPRI-SOG ESTs. Therefore, RAI 02.05.02-5 is resolved and closed.

**Update of EPRI-SOG Seismic Source Models**

FSAR Sections 2.5S.2.4.2 through 2.5S.2.4, present the applicant's sensitivity studies to determine whether the 1986 EPRI-SOG seismic source models needed to be updated. This determination is based on the availability of more recent seismic hazard studies and data for the region surrounding the STP site. The applicant assessed the need for updates after evaluating: (1) the updated earthquake catalog and resulting changes in the rate of earthquake occurrence as a function of magnitude, (2) changes in the maximum magnitude distributions for seismic sources, and (3) possible newly identified seismic sources in the region surrounding the site.

<u>Update of Seismicity Parameters</u>. FSAR Subsection 2.5S.2.4.2, describes the applicant's assessment of the updated earthquake catalog for the region surrounding the site relative to two key areas. First, the applicant assessed the effect of the new earthquake data (see Subsection 2.5S.2.4.1 above) on earthquake recurrence estimates for seismic sources to the north and west of the site. Second, the applicant estimated seismicity parameters for the EPRI-SOG EST sources south and east of the site that extend into the Gulf of Mexico and adjacent on-shore areas, which were not fully developed by the original EPRI-SOG ESTs.

For the seismic sources to the north and west of the site, the applicant used the updated earthquake catalog to estimate updated earthquake recurrence rates for comparison with those developed by the EPRI-SOG ESTs for the original EPRI-SOG source models. The applicant found that the updated recurrence rates are about four percent higher than those originally estimated by the EPRI-SOG ESTs. The applicant concluded that this difference is insignificant. Because of the relatively small difference between the updated and original recurrence rates, the staff concurred with the applicant's decision to use the original recurrence rates for seismic sources to the north and west of the site.

For the seismic sources south and east of the site that extend into the Gulf of Mexico and adjacent on-shore areas, the applicant calculated new seismicity parameters for each degree cell within these sources, because they were not developed in the original EPRI-SOG source models. Rather than assessing the sensitivity of these new seismicity parameters for the final hazard results, these updates were directly incorporated by the applicant into the seismic hazard analysis for the site. Because these earthquake occurrence parameters were not developed by the ESTs in the original EPRI-SOG seismic source models, the staff concurred with the applicant's decision to incorporate this new information into the seismic hazard analysis for the site.

New Maximum Magnitude Information. Based on the geological and seismological data published since the 1986 EPRI-SOG seismic source model, the applicant evaluated whether the maximum magnitudes for the EPRI-SOG sources needed to be updated. As a result of the two 2006 Gulf of Mexico earthquakes, the applicant determined that there was a need to update the EPRI-SOG seismic source models for the Gulf of Mexico.

Gulf of Mexico

Both the February 10, 2006, magnitude 5.1 (M) earthquake and the September 10, 2006, magnitude 5.8 (M) earthquake were in the Gulf of Mexico. As a result of these earthquakes, the applicant updated five of the six EPRI-SOG EST Gulf Coast source maximum magnitude distributions.

The staff issued RAI 02.05.02-13, asking the applicant for additional details regarding the updated maximum magnitude distributions for the Gulf Coast seismic source zones. Specifically, the staff asked whether the applicant had used the expert elicitation process described in NUREG/CR–6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts," referred to as the "SSHAC process." In its response to RAI 02.05.02-13, dated September 4, 2008 (ML092530449), the applicant stated that the updated maximum magnitude distributions for the Gulf Coast seismic sources used a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 study. The response included a description of the SSHAC Level 2 study that identifies the technical integrators (TIs), the resource and proponent experts, and the participatory peer review panel. There is also a general description of the expert elicitation process and the outcome of the process, which was to update the maximum magnitude values for the Gulf Coast sources.

Because the applicant's response to RAI 02.05.02-13, provided only a general description of the SSHAC Level 2 process and the updated maximum magnitude values, the staff issued RAI 02.05.02-21, asking the applicant for more details. Specifically, the staff wanted to know how the experts' opinions are integrated into the development of the final Gulf Coast source models, how any conflicting opinions between the experts were handled, and how the final source models represent an informed consensus of the community. In its response to RAI 02.05.02-21, dated September 21, 2009 (ML092710096), the applicant provided considerably more details about the SSHAC Level 2 study. Specifically, the applicant cites three specific questions that focus on the SSHAC Level 2 study:

(1) Does the Gulf of Mexico seismicity, and in particular the February and September earthquakes, provide evidence that EPRI-SOG Gulf Coast Source Zone (GCSZ) characterizations need to be updated?

(2) What components of the characterizations (e.g., geometry, recurrence, $M_{max}$) need to be updated?

(3) What methodology should be used to update those components?

The applicant's response also refers to interviews with numerous experts in order to determine the range of interpretations among the informed technical community, which is one of the main goals of the SSHAC process. Because the two 2006, Gulf of Mexico earthquakes were a main impetus for updating the EPRI-SOG GCSZ, these interviews focused on determining whether the experts were familiar with the two earthquakes and whether they knew of any distinguishable geologic features or structures that may have been sources for the earthquakes. The applicant noted that the interviews demonstrated no consensus among the informed technical community as to whether there is a distinguishable geologic feature or structure associated with either of the 2006, Gulf earthquakes.

As a result of these expert solicitations, the applicant's TIs determined that: (1) the geometry of the EPRI-SOG GCSZ does not need to be updated; (2) only the maximum magnitude distributions for the GCSZ should be updated; and (3) there is insufficient evidence to develop a new seismic source.

Regarding the first and third conclusions, the TIs determined that if the 2006, earthquakes could be related to a specific structure, then a source zone local to the earthquakes and encompassing the structure would be the best representation of the potential hazard. The TIs also agreed that if the earthquakes could not be related to a specific structure, the best representation of the potential hazard would be to allow similar earthquakes to occur anywhere within the Gulf of Mexico. After evaluating the available data and the existing GCSZ characterizations, the applicant stated that the TIs determined that the existing EPRI-SOG GCSZ geometries "adequately characterize both options and thus capture the 'legitimate range of technically supportable interpretations among the entire informed technical community'" (NUREG/CR–6372, page 6). SER Figure 2.5S.2-6 shows the EPRI-SOG GCSZ geometries along with the epicenters for the two Gulf earthquakes.

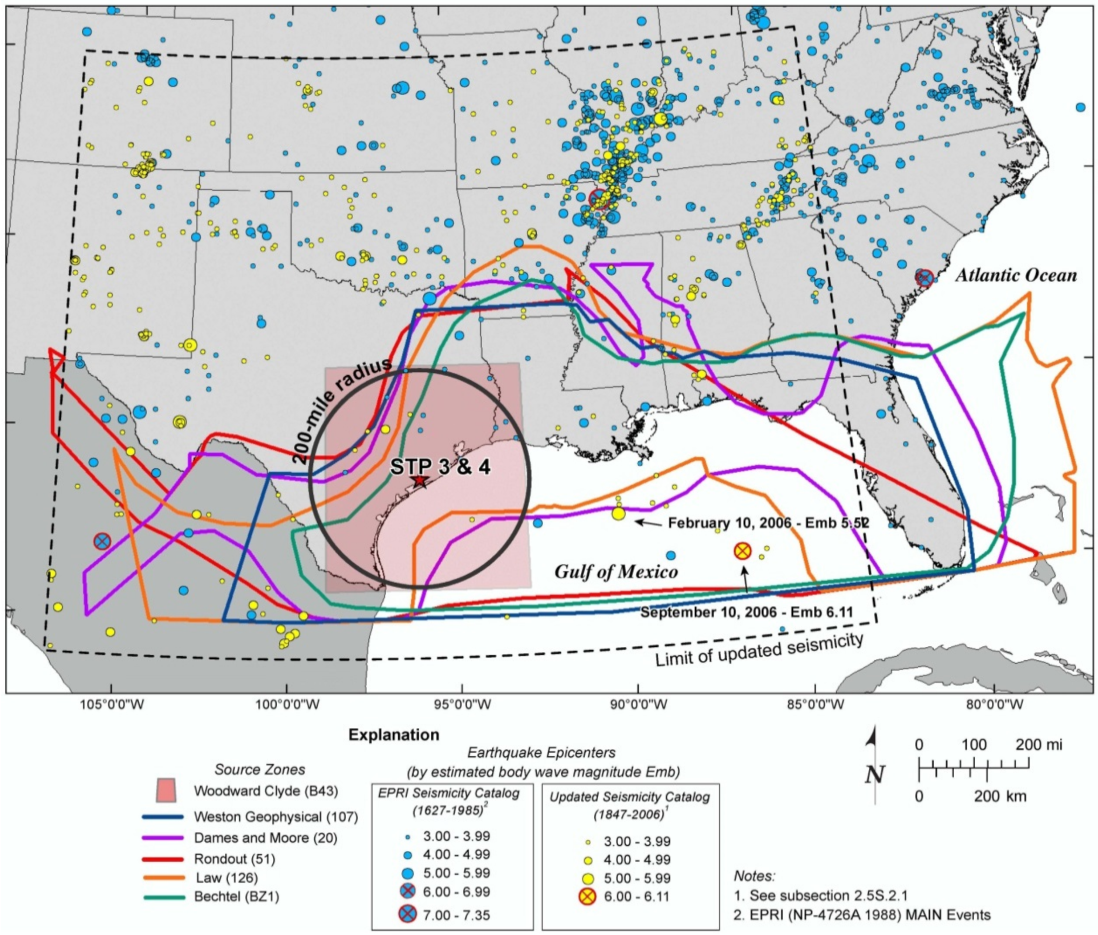


**Figure 2.5S.2-6 EPRI EST Gulf Coast Background Source Zones (FSAR Figure 2.5S.2-8)**

With regard to the September 2006, magnitude 5.8 (M**)** earthquake, three of the EPRI-SOG GCSZs include the September 2006, epicenter and thus represent the “interpretation that an earthquake similar to the September event can occur anywhere within the Gulf of Mexico.” Also, three of the GCSZs do not include the epicenter and thus represent the “interpretation that the earthquake is related to a source local to the epicenter and outside the existing source zones.” With regard to the February 2006 magnitude 5.1 (M) earthquake, the TI team evaluated the hypothesis proposed by some of the experts that the earthquake was caused by a large-scale landslide on the Sigsbee escarpment. The TI team concluded that the existing EPRI-SOG GCSZ geometries capture this hypothesis in addition to other potential sources, such as the earthquake occurring in the basement beneath the sedimentary section.

The final determination of the TI team is the need to only update the maximum magnitude distributions of the EPRI-SOG GCSZ. The TIs updated the maximum magnitude distribution of each EPRI-SOG GCSZ with the following magnitudes and weights: $m_b$ 6.1 [0.1], 6.6 [0.4], 6.9 [0.4], and 7.2 [0.1]. After the TI team presented these conclusions to the SSHAC peer review panel, the peer review panel concurred with the TI team that only the maximum magnitude distributions of the EPRI-SOG GCSZ needed to be updated, but the panel disagreed with the maximum magnitude distribution (magnitudes and weights) shown above. As a basis for this conclusion, the applicant stated that the SSHAC peer review panel “did not think it was appropriate to base the updated $M_{max}$ distributions for each EST on the USGS National Hazard Map source characterizations." The USGS uses only a single maximum magnitude value of $m_b$ 7.2 for its extended margin source zone, which covers the region surrounding the STP site. As a result of the SSHAC peer review panel recommendation, the TI team revised the maximum magnitude distributions for each of the six EST GCSZ models on an individual basis. Two of the six maximum magnitude distributions include $m_b$ 7.2 as the upper end of their distributions.

**Table 2.5S.2-3 Updated Maximum Magnitude ($M_{max}$) Distributions ($m_b$) for the Gulf Coast Seismic Source Zones**

| EST | Original $M_{max}$ | TI Team Initial $M_{max}$ | Final $M_{max}$ |
|---|---|---|---|
| Bechtel (BZ1) | 5.4[0.1], 5.7[0.4], 6.0[0.4], 6.6[0.1] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | 6.1[0.1], 6.4[0.4], 6.6[0.5] |
| D & M (Zone 20) | 5.3[0.8], 7.2[0.2] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | 5.5[0.8], 7.2[0.2] |
| Law (Zone 126) | 4.6[0.9], 4.9[0.1] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | 5.5[0.9], 5.7[0.1] |
| Rondout (Zone 51) | 4.8[0.2], 5.5[0.6], 5.8[0.2] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | 6.1[0.3], 6.3[0.55], 6.5[0.15] |
| Weston (Zone 107) | 5.4[0.71], 6.0[0.29] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | 6.6[0.89], 7.2[0.11] |
| WCC (B43) | 4.9[0.17], 5.4[0.28], 5.8[0.27], 6.5[0.28] | 6.1[0.1], 6.6[0.4], 6.9[0.4], 7.2[0.1] | No update |
| D & M=Dames and Moore, EST=Earth Science Team, WCC=Woodward-Clyde Consultants. | | | |

The TI team did not update the maximum magnitude distribution for the Woodward-Clyde Consultants source zone B43 because this source zone is for the Central United States and its southern boundary is 273 km (170 mi) and 635 km (395 mi) from the two 2006, Gulf Coast earthquakes.

Regarding the applicant's response to RAI 02.05.02-21, the staff concurred with the applicant's assertion of legitimately following an SSHAC Level 2 process to update the EPRI-SOG GCSZ models. The SSHAC expert elicitation process is recommended in RG 1.208 as an acceptable way to update pre-existing seismic source models. Furthermore, as part of the staff's review of previous ESP applications, the staff reviewed several SSHAC Level 2 seismic source updates for important seismic sources in the CEUS—such as New Madrid, Charleston, and Wabash Valley. The staff concluded that the applicant has solicited an adequate number of experts and that the range of technical opinions adequately represents the legitimate range of technically supportable interpretations. The staff also found that the TI team had adequately incorporated the range of expert opinions in the decision to maintain the original EPRI-SOG GCSZ geometries and to update the maximum magnitude distribution for each source. The staff also concurred with the applicant's conclusion that there is too much uncertainty regarding the sources of the two 2006, Gulf of Mexico earthquakes to support the development of a new seismic source zone for these two sources and that, as such, the original EPRI-SOG GCSZ geometries adequately characterize the seismic hazard along the Gulf Coast as well as within the Gulf of Mexico. However, the staff was concerned with the TI team's decision (and by default the applicant's decision) to revise the original maximum magnitude distribution for the GCSZ models based on the SSHAC peer review panel's recommendations. Specifically, for the final maximum magnitude distributions, only two of the six EPRI-SOG $M_{max}$ distributions extend out to the USGS value of $m_b$ 7.2. The weight placed on this upper end value of $m_b$ 7.2 is open to interpretation based on the TI team's belief concerning how much the USGS $M_{max}$ value of $m_b$ 7.2 is supported by the informed technical community.

To resolve the staff's concern with regard to the different $M_{max}$ distributions and their impact on the overall seismic hazard at the STP site, the applicant conducted a sensitivity test by incorporating the TIs initially proposed $M_{max}$ distributions into the GCSZ models. The applicant's sensitivity test covered three different $M_{max}$ update scenarios. Scenario 1 incorporated the TIs initial $M_{max}$ distribution for three of the ESTs' GCSZs (specifically, Bechtel BZ1, Rondout 51 and Weston 107 seismic sources). These three are the sources that include the two 2006 earthquake epicenters. Scenario 2 incorporated the TIs initial $M_{max}$ distribution for two additional teams (Dames and Moore 20 and Law 126), even though these sources did not cover the two epicenters. Finally, Scenario 3 included a model that incorporated the TIs initial $M_{max}$ distribution for all six EPRI teams' models (see SER Table 2.5S.2-4).

**Table 2.5S.2-4 Maximum Magnitude Distributions and Scenarios Used in the Sensitivity Test**

| Gulf Coast Source Zone | EPRI-SOG Original Mmax | STP Updated Mmax FSAR Table 2.5S.2-13 | Case 1 Mmax | Case 2 Mmax | Case 3 Mmax |
|---|---|---|---|---|---|
| Bechtel Group - BZ1 | 5.4 [0.1]<br>5.7 [0.4]<br>6.0 [0.4]<br>6.6 [0.1] | 6.1 [0.10]<br>6.4 [0.40]<br>6.6 [0.50] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |
| Dames & Moore - 20 | 5.3 [0.8]<br>7.2 [0.2] | 5.5 [0.80]<br>7.2 [0.20] | 5.5 [0.80]<br>7.2 [0.20] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |
| Law - 126 | 4.6 [0.9]<br>4.9 [0.1] | 5.5 [0.90]<br>5.7 [0.10] | 5.5 [0.90]<br>5.7 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |
| Rondout - 51 | 4.8 [0.2]<br>5.5 [0.6]<br>5.8 [0.2] | 6.1 [0.30]<br>6.3 [0.55]<br>6.5 [0.15] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |
| Weston - 107 | 5.4 [0.71]<br>6.0 [0.29] | 6.6 [0.89]<br>7.2 [0.11] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |
| Woodward Clyde - B43 | 4.9 [0.17]<br>5.4 [0.28]<br>5.8 [0.27]<br>6.5 [0.28] | 4.9 [0.17]<br>5.4 [0.28]<br>5.8 [0.27]<br>6.5 [0.28] | 4.9 [0.17]<br>5.4 [0.28]<br>5.8 [0.27]<br>6.5 [0.28] | 4.9 [0.17]<br>5.4 [0.28]<br>5.8 [0.27]<br>6.5 [0.28] | 6.1 [0.10]<br>6.6 [0.40]<br>6.9 [0.40]<br>7.2 [0.10] |

The results of the sensitivity study demonstrated that the increase in the GMRS at the STP site resulting from these three different scenarios ranged from 0 percent to 11 percent, with the exception of Scenario 3, which increased the GMRS by as much as 18% at a spectral frequency of 25 Hz. However, Scenario 3 includes an $M_{max}$ update to the Woodward Clyde B43 source zone, which is located very far from the two 2006, Gulf earthquakes (273 km (170 mi) and 635 km (395 mi), and therefore, Scenario 3 is not considered plausible by the staff. The GMRS resulting from the three scenario $M_{max}$ updates are shown below in SER Figure 2.5S.2-7.

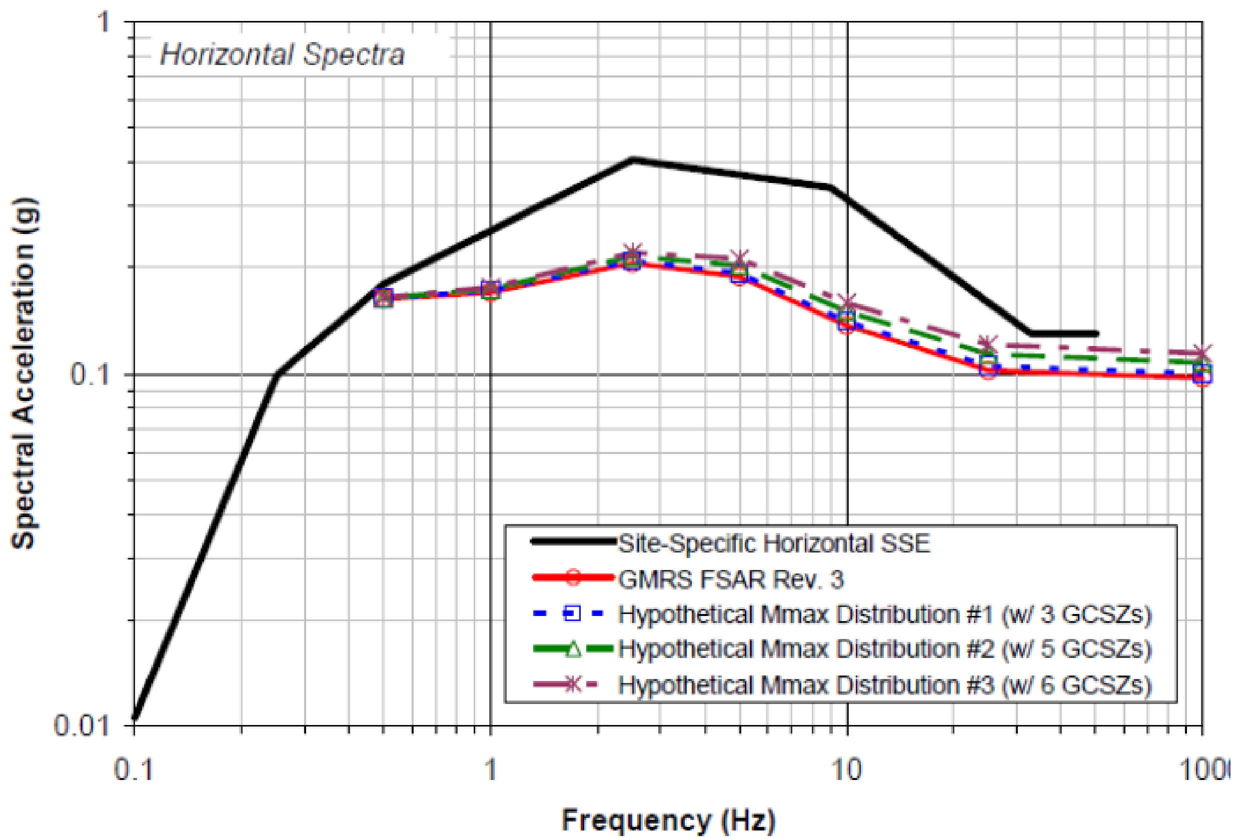


**Figure 2.5S.2-7 Response Spectra from the Sensitivity Test and Comparison to the Site-Specific SSE**

Based on the results of the applicant's sensitivity study, the staff concluded that the higher $M_{max}$ distribution originally proposed by the TI team does not significantly increase the GMRS for the STP site. Under either Scenario 1 or 2, the increase in the GMRS is only about 10 percent at most. This result can be attributed to the significant epicentral distances of the two 2006 Gulf earthquakes from the STP site as well as the relatively sparse seismicity within the Gulf. The SSHAC expert elicitation process specifies that the center, body, and range of the informed technical community should be represented when modeling seismic source zones for PSHA studies. Scenario 1 of the sensitivity study achieves this SSHAC goal better than either Scenarios 2 or 3, while simultaneously reaching an $M_{max}$ value of $m_b$ 7.2 for four of the six ESTs. Under Scenario 1, the largest increase in the GMRS for the STP site is only 3 percent. For the actual maximum magnitude distributions adopted by the TI team, only the Law Engineering

Source Zone-126 is somewhat low ($m_b$ 5.5 [0.9], 5.7 [0.1]); however, this source zone does not include either of the two 2006 Gulf Coast earthquakes.

Therefore, the staff concluded that the revised EPRI-SOG maximum magnitude distributions adequately characterize the seismic hazard potential of the Gulf Coast region with respect to the STP site.

As such, the staff considers the GCSZ $M_{max}$ update issue resolved. Therefore, RAI 02.05.02-13 and RAI 02.05.02-21 are resolved and closed. The staff acknowledges that other applications influenced by the Gulf Coastal seismic sources could reference the sensitivity test results provided in response to RAI 02.05.02-21, for the STP site. However, each application site will have a different spatial position with respect to the EPRI-SOG source zones, and as such, the applicant's sensitivity study does not have generic significance.

<u>Non-Gulf Seismic Sources</u>

In addition to the Gulf of Mexico seismic sources, the staff also reviewed the adequacy of the maximum magnitudes selected by the EPRI-SOG ESTs for the non-Gulf seismic sources. The staff issued RAI 02.05.02-24, requesting information from the applicant about the low maximum magnitudes and probabilities of activity (Pa) for seismic sources located in the northwest corner of the site region. In its response to RAI 02.05.02-24, dated September 21, 2009 (ML092710096), the applicant updated FSAR Figure 2.S.2-8, "EPRI EST Gulf Coast Background Source Zones," to show all of the EPRI-SOG seismic sources that fall within the 320-km (200-mi) region surrounding the site, and not only those that contribute at least 99 percent to the total hazard. In its response also adds the Dames and Moore EST New Mexico source, the Rondout EST Background 50 source, and the Weston EST Combination Zone 109 to FSAR Figure 2.S.2-8 and to the seismic sources in FSAR Tables 2.5S.2-7 through 2.5S.2-12. The applicant concluded by stating, "the composite EST seismic sources, which cover the northwest portion of the site region, do adequately characterize the low contribution to seismic hazard from this area." However, the applicant did not adequately address the staff's specific concerns with regard to the low Pa and maximum magnitude values for some of the sources in the northwest corner of the site region. Specifically, the Bechtel Group EST assigned a Pa of only 0.1 and $M_{max}$ values ranging from $m_b$ 5.4 to 6.6 for their Texas Platform source. And the Dames and Moore EST assigned a Pa of 0.35 and $M_{max}$ values of 5.5 [0.8] and 7.2 [0.2] for their Ouachitas Fold Belt. The staff further communicated with the applicant on this issue. The applicant indicated that the Pa value for Bechtel BZ2 was intended to be taken directly from original EPRI-SOG model, but was incorrectly transcribed to FSAR Table 2.5S.2-7 as 0.1, instead of the correct value of 1.0. The applicant explained that this inconsistency is a typographical error and the correct value of 1.0 was used in the seismic hazard calculation. Furthermore, the applicant addressed the Dames and Moore source zone C08 and 25 low Pa values. According to the applicant, the two zones are exclusive, however, the probability for the two zones are respectively 0.35 and 0.65. Therefore, the total probability is 1.0 for the same geographic area. Based on these clarifications, RAI 02.05.02-24 is resolved and closed.

<u>New Seismic Source Zones</u>. FSAR Subsection 2.5S.2.4.4, describes the applicant's evaluation of seismic sources that were not included in the original EPRI-SOG source models for the region surrounding the STP site, such as the MEEG and the NMSZ. Although the EPRI-SOG EST had developed source models for these two sources, the original EPRI-SOG "screening" for the STP site, which was performed in 1989, did not include them because they were either: (1) too distant, (2) not large enough in magnitude, or (3) too infrequent. In addition to the MEEG

and NMSZ, the staff also identified other potential seismic source zones that were not among those selected by the applicant from the EPRI-SOG seismic sources. These additional seismic sources are the MAT and the Saline River Source. Further discussion of these source zones is presented below.

Mt. Enterprise-Elkhart Graben

FSAR Subsection 2.5S.2.4.4.1, describes the MEEG as a system of roughly east-west striking normal faults of various lengths and widths. The most recent movement on the faults that compromised the MEEG system was "likely Eocene [37.2 to 58.7 mya] in age or younger." The applicant also states that several publications document Quaternary motion and active creep along the MEEG. The applicant postulates that this motion may be driven by movement of salt at depth, because the MEEG "is rooted in the Jurassic Louann Salt at maximum depths of 4.5 to 6 km." The applicant concluded that because creep across the MEEG is driven by movement of salt at depth, "the fault is not accommodating tectonic deformation and thus is not an independent source of moderate to large earthquakes."

The staff issued RAI 02.05.02-14, requesting the applicant to justify why the nature of the loading mechanism (salt movement rather than tectonic forces) disqualifies the MEEG as a seismic source. In its response to RAI 02.05.02-14, dated September 4, 2008 (ML082530449), the applicant stated that in the probabilistic seismic hazard assessment for STP, Units 3 and 4, the MEEG is not disqualified from being a seismic source based upon its loading mechanism or any other factor. The applicant provided more details as to why the MEEG is not likely to accumulate the stress and elastic strain energy required for a seismogenic rupture. Specifically, the applicant described the MEEG as shallow, crustal, listric normal faults that root into the salt and do not penetrate into the underlying crystalline basement. Faults of this style are considered to be aseismic. The staff reviewed the applicant's response to RAI 2.5.2-14, and concluded that the applicant has adequately evaluated the MEEG as a potential seismic source. The staff concurred with the applicant's conclusion that the MEEG is unlikely to generate large earthquakes. As described in FSAR Subsection 2.5S.2.4.4.1, the applicant evaluated the creep across the MEEG and developed a $M_{max}$ distribution of $m_b$ 6.0 (0.2), 6.5 (0.6), and 6.6(0.2). The applicant then evaluated the contribution of the MEEG source to the total seismic hazard. The applicant concluded that the MEEG had contributed less than 1 percent to the total hazard and should therefore not be included. Based on the distance of the MEEG to the site (about 320 km [200 mi]) and the aseismic slip as a result of salt movement, the staff concurred with the applicant's decision and RAI 02.05.02-14 is resolved and closed.

New Madrid Seismic Zone

The NMSZ extends from southeastern Missouri to southwestern Tennessee and is located more than 800 km (500 mi) northeast of the STP, Units 3 and 4, site. The NMSZ produced a series of large-magnitude earthquakes between December 1811, and February 1812. Paleoliquefaction studies in the region of the 1811–1812, New Madrid earthquakes have identified several sequences of prehistoric earthquakes that have led researchers to estimate a mean recurrence interval of approximately 500 years for these earthquake sequences. Because the mean recurrence interval represents a higher activity rate than was modeled by the EPRI-SOG ESTs, the applicant updated the NMSZ source model. For this update, the applicant incorporated the NMSZ source model described in Exelon's ESP application for the Clinton (Illinois) site. The applicant's sensitivity study of the updated NMSZ source model showed that it is a significant contributor to the total hazard and, therefore, the applicant included the updated NMSZ in the

PSHA. Based on the applicant's incorporation of the New Madrid source model developed by Exelon for the Clinton ESP, the staff concluded that the applicant has adequately modeled the NMSZ.

Middle America Trench

The MAT is a major subduction zone off the southwestern coast of Middle America stretching from Central America to Costa Rica. The trench is 2,750 km (1,700 mi) long and 6,669 m (21,880 ft) at its deepest point. The largest earthquake in this century from the MAT is the 1985 Michoacan earthquake, which had a magnitude of 8.0 (M**)**. The MAT is about 1,300 km (800 mi) from the STP site, and the 1985 earthquake was felt at several locations in Texas. The staff issued RAI 02.05.02-9, asking the applicant to describe the potential hazard to the STP site. In its response to RAI 02.05.02-9, dated July 24, 2008 (ML082100162), the applicant pointed to a sensitivity study in effect at the time to address this issue and identifies NRC commitment COM 2.5S-1. The staff then issued RAI 02.05.02-20, requesting details of the applicant's sensitivity study. In its response to RAI 02.05.02-20, dated July 20, 2009 (ML092030132), the applicant described the source model developed for the MAT and the process used to develop the MAT model. Also, the applicant's SSHAC Level 1 process included modeling (epistemic) uncertainty in the source segmentation, as well as geometry, rupture, convergence rate, and magnitude. The applicant's final model for the MAT contained two independent sources, which resulted in five rupture scenarios. The applicant also considered the potential for a multiple segment rupture of the MAT. In addition to developing the MAT source model, the applicant also developed a ground motion prediction equation appropriate for the MAT source. The result is a seismic hazard curve for the MAT source that falls well below the total hazard curve for the STP site. Therefore, the applicant did not include the MAT source in the PSHA for the STP site. After reviewing the applicant's sensitivity study, the staff concluded that the applicant has adequately modeled the MAT source for the STP site, and the MAT does not contribute significantly to the overall hazard at the STP site. Therefore, RAI 02.05.02-9 and RAI 02.05.02-20 are resolved and closed.

Saline River Seismic Source

Several paleoliquefaction features in southeastern Arkansas and northeastern Louisiana indicate that previously unrecognized seismic source(s) may exist in those areas. The staff issued RAI 02.05.02-15, requesting the applicant to explain whether there is an evaluation of the potential seismic hazard of this source, commonly referred to as the Saline River seismic source. In its response to RAI 02.05.02-15, dated September 24, 2008 (ML082530449), the applicant stated that all of the paleoliquefaction features are within 175 km (109 mi) of the NMSZ and are likely attributed to that source. In addition, the applicant noted that local sources that are proximal to the paleoliquefaction features have been hypothesized by researchers. The magnitudes (M between 5.5 and 6.5) and distance of the source to the site (more than 675 km [419 mi]) imply that these sources will not have a significant impact on the total hazard. The staff issued RAI 02.05.02-22, requesting the applicant to clarify whether magnitude 6 and above earthquakes from the Saline River source could potentially contribute significantly to the overall site seismic hazard. In its response to RAI 02.05.02-22, dated September 21, 2009 (ML092710096), the applicant refered to the Cox et al. (2004) study, which hypothesized that if a single earthquake can caused all of the paleoliquefaction features in the two field areas, the estimated magnitude would be about 6.5 (M**)**. Cox et al. (2004) considers the hypothesis of a single earthquake less likely than multiple events as the source for the paleoliquefaction features. The applicant restated the conclusion that the large distance from the STP site makes this potential source a very unlikely significant contributor to the total hazard. After reviewing the applicant's responses to RAI 02.05.02-15 and RAI 02.05.02-22, the staff concurred with the applicant that the moderate size of the postulated earthquakes (up to 6.5 M**)** and the significant distance from the site (over 675 km [419 mi]) imply that the Saline River paleoliquefaction features do not represent a seismic source that would contribute significantly to the overall hazard of the STP site. Therefore, RAI 02.05.02-15 and RAI 02.05.02-22 are resolved and closed.

**Staff Conclusions of the Geologic and Tectonic Characteristics of the Site and Region**

After reviewing STP COL FSAR Subsections 2.5S.2.2 and 2.5S.2.4, the staff concluded that, the applicant has adequately updated the original EPRI-SOG seismic source models as the input to the PSHA for the STP site. In addition, the staff concluded that the applicant has adequately considered seismic sources that were not part of the EPRI-SOG sources for the STP site, such as the MEEG, MAT, Saline River, and NMSZ. The staff found that the applicant's use of the EPRI-SOG seismic source model and the updates of the model, as described by the applicant in FSAR Subsections 2.5S.2.2 and 2.5S.2.4, form an adequate basis for the seismic hazard characterization of the site that meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

2.5S.2.4.3 Correlation of Earthquake Activity with Seismic Sources

FSAR Subsection 2.5S.2.3, describes the correlation between the updated seismicity and the EPRI-SOG seismic source model. The applicant presented comparative figures (FSAR Figures 2.5S.2-1 through 2.5S.2-6) showing differences between the original EPRI-SOG earthquake catalog and the updated earthquake catalog. The applicant compared the distribution of earthquake epicenters in both the original EPRI-SOG historical catalog (1627-1984) and the updated earthquake catalog (1985–2006), with the seismic sources characterized by the 1986 EPRI-SOG Project. The applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure, and there are no clusters of seismicity suggesting a new seismic source that was not captured by the EPRI-SOG seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require a significant revision to the geometry of any of the EPRI-SOG seismic sources. The applicant based these conclusions on a comparison of the distribution of earthquake epicenters in both the original EPRI-SOG historical catalog and the updated seismicity catalog with the seismic sources characterized by the EPRI-SOG.

However, earthquakes that occurred within the Gulf of Mexico and Coastal Region in 2006 prompted the applicant to update the seismic source parameters (i.e., $M_{max}$, activity rate, b value, and source geometries) of the Gulf of Mexico seismic source zones defined by the EPRI-SOG seismic source model.

Based on the spatial distribution of earthquakes in the updated catalog, the staff concurred with the applicant's conclusion that significant revisions to the existing EPRI-SOG source geometries are not warranted. The staff's review evaluated the completeness of the applicant's updated earthquake catalog and the applicant's subsequent conclusions, by comparing the applicant's earthquake catalog to a compilation catalog derived from the USGS seismicity catalogs. The catalog data from February 1985, through September 2006, are depicted in SER Figure 2.5S.2-8 as the red circles. The applicant's updated seismicity catalog is illustrated by the blue circles, which cover February 1985, through September 2006. The comparison of these data sets illustrates that the applicant's updated earthquake catalog adequately characterizes the seismicity within and around the STP, Units 3 and 4, site region. Because the applicant's earthquake catalog is complete through 2006, the staff also determined whether there has been any significant seismicity since 2006, that would change the applicant's conclusions. The yellow circles in SER Figure 2.5S.2-8, illustrate the seismicity documented in the USGS catalog from September 2006, through November 2009. This recent seismicity does not show any significant deviations from the applicant's seismicity catalogs.

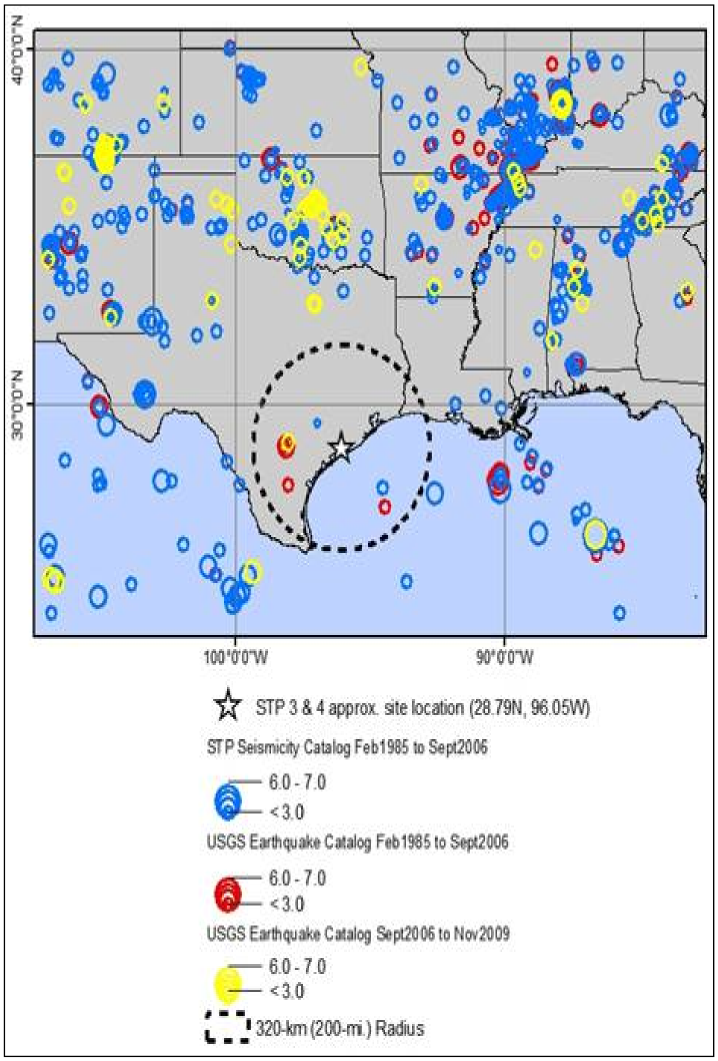


**Figure 2.5S.2-8 A Comparison of Events ($m_b \geq 3$) from the STP Units 3 and 4 Site Updated Earthquake Catalog from 1985 to 2006 (Blue Circles), the USGS Earthquake Catalog from 1985 to 2006 (Red Circles), and the USGS Earthquake Catalog from 2006 to 2009 (Yellow Circles); (The Star Corresponds to the Location of the STP Units 3 and 4 Site and the Dashed Black Oval Corresponds to the 320-km (200-mi) Site Radius)**

Therefore, the staff concluded that the STP, Units 3 and 4, earthquake catalog adequately characterizes regional and local seismicity through November 2009. In addition, the staff agreed that the spatial distribution of earthquakes in the region had not changed significantly since the publication of the EPRI-SOG earthquake catalog. The staff found that the applicant has adequately evaluated the potential for new seismic sources or for revisions to existing source geometries based on seismicity patterns. Therefore, the applicant meets the requirements of 10 CFR 52.79 and 10 CFR 100.23.

2.5S.2.4.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

FSAR Subsection 2.5S.2.4, presents the earthquake potential for the STP, Units 3 and 4, sites in terms of the controlling earthquakes. The applicant determined the high- and low-frequency controlling earthquakes by deaggregating the PSHA results at selected probability levels. Before determining the controlling earthquakes, the applicant updated the 1989 EPRI-SOG PSHA. For the update, the applicant used the seismic source zone adjustments described in SER Subsections 2.5S.2.2.2 and 2.5S.2.2.4, and the new ground motion models described in SER Subsection 2.5S.2.2.4.

The staff's review focused on FSAR Subsection 2.5.2.4, which includes the applicant's updated PSHA and the STP, Units 3 and 4, site controlling earthquakes, which the applicant determined after completing the PSHA. SER Subsection 2.5S.2.4.2, describes the staff's review of the applicant's updated EPRI-SOG seismic source model. This SER section focuses on the review of the application of the updated seismic source model to the hazard calculation at the STP, Units 3 and 4, sites.

**PSHA Calculation**

In FSAR Subsection 2.5S.2.4.1, the applicant stated that the PSHA calculation used the Risk Engineering, Inc., FRISK88 seismic hazard software. This software is different from the software used in the original 1989 EPRI-SOG PSHA calculation. For this reason, to ensure that the new software could accurately reproduce the 1989 EPRI-SOG PSHA results, the applicant first performed a PSHA using the original 1989 EPRI-SOG primary seismic sources and ground motion models. In FSAR Table 2.5S.2-14, "Comparison of EPRI (Reference 2.5S.2-16) and current hazard results for Bechtel Group EST using EPRI (Reference 2.5S.2-16) assumptions," the applicant compared the results from FRISK88 with the original EPRI-SOG hard rock results from the Bechtel Group EST and concluded that the differences in hazard are small (i.e., less than eight percent). The applicant also stated that the results of this software validation are different depending on the EPRI-SOG EST. However, it only presented numerical comparisons for the Bechtel EST. Thus, the staff issued RAI 02.05.02-11, requesting the applicant to provide the results of the software validation for all of the EPRI-SOG ESTs.
In its response to RAI 02.05.02-11, dated August 12, 2008 (ML082270381), the applicant provided the software comparison results for all of the EPRI-SOG ESTs. The applicant noted significant differences in the validation results for all of the ESTs except Bechtel and Weston. The applicant stated that the hazard calculations using its software are significantly lower than the original EPRI-SOG calculations for the following ESTs: Law (mean, 15$^{th}$, 50$^{th}$, and 85$^{th}$ fractile hazard); Rondout (15$^{th}$ fractile hazard); and Woodward-Clyde (15$^{th}$ fractile hazard). For these ESTs, the host source zones had $M_{max}$ distributions that extended below $m_b$ 5.0. The applicant attributes the differences to undocumented assumptions in the EPRI-SOG analysis regarding the maximum magnitude values for these source zones. For the Dames and Moore team, the applicant observed significantly larger values for the 85$^{th}$ fractile hazard using its software. The applicant observed that the Dames & Moore team used a "no-smoothing" assumption for the seismicity parameters of sources 20, 25, and C08. A lack of historical seismicity means that no seismicity parameters were estimated in the degree cells near the site.

The applicant attributes the differences to undocumented assumptions (related to smoothing) in the original EPRI-SOG PSHA.

The staff reviewed the applicant's response to RAI 02.05.02-11, and concluded that the PSHA software has accurately reproduced the original 1989 EPRI-SOG PSHA calculation based on comparisons of the Bechtel and Weston hazard curves. The staff concluded that the undocumented assumptions related to maximum magnitude and smoothing in the original EPRI-SOG PSHA calculation and the resulting hazard curve differences observed for the remaining ESTs are not significant. In the updated PSHA, the applicant increased $M_{max}$ values for all seismic sources above 5.0, as an overall update. The applicant also recalculated seismicity parameters for all degree cells adjacent to the site using the updated seismicity catalog. The staff found these changes acceptable and RAI 02.05.02-11 is resolved and closed.

**Controlling Earthquakes**

FSAR Subsection 2.5S.2.4.4.5, describes the deaggregation of final PSHA hazard curves to determine the controlling earthquakes for the STP, Units 3 and 4, sites. To determine the low- and high-frequency controlling earthquakes, the applicant followed the procedure outlined in Appendix D to RG 1.208. This procedure specifies that the controlling earthquakes are determined from the deaggregation of the PSHA results corresponding to the annual frequencies of $10^{-4}$, $10^{-5}$, and $10^{-6}$, which are based on the magnitude and distance values that contribute most to the hazard at the average of 1 and 2.5 Hz and the average of 5 and 10 Hz. SER Table 2.5S.2-2 (reproduced from FSAR Table 2.5S.2-17, "Controlling Magnitudes and Distances from Deaggregation"), lists the low- and high-frequency controlling earthquakes for the STP, Units 3 and 4, sites. For the high-frequency mean $10^{-4}$ and $10^{-5}$ hazard levels, the controlling earthquakes are a M 6.1 at 46 km (29 mi) and a M 6.7 at 230 km (143 mi), respectively, corresponding to earthquakes from local seismic source zones. In contrast, for the low-frequency mean $10^{-4}$ and $10^{-5}$ hazard levels, the controlling earthquakes are a M 7.6 at 880 km (547 mi) and a M 7.7 at 890 km (553 mi), respectively. These controlling earthquakes correspond to an event in the NMSZ. After reviewing these four controlling earthquake magnitudes and distances, the staff concluded that they are representative of earthquakes in the site region and adequately characterize the seismic hazard for the site.

2.5S.2.4.5 Seismic Wave Transmission Characteristics of the Site

FSAR Subsection 2.5S.2.5, describes the method used by the applicant to develop the STP, Units 3 and 4, site free-field ground motion spectra. The seismic hazard curves generated by the applicant's PSHA are defined for generic hard rock conditions (characterized by an S-wave velocity of 2.8 km/s (9,200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 9,144 m (30,000 ft) below the ground surface at the STP, Units 3 and 4, sites. To determine the site free-field ground motion, the applicant performed a site-response analysis. The output of the applicant's site-response analysis is site-specific amplification function, which is then used to determine the UHRS for the $10^{-4}$ and $10^{-5}$ hazard levels. These UHRS are then used to calculate the GMRS for the site.

**Site Response Inputs**

An important part of the site-response analysis is the model of the site subsurface soil and rock properties. Key properties include the S-wave velocities and the strain dependent behavior of each of the soil layers underlying the site. To model the strain dependent behavior of the soil in the upper 182 m (600 ft), the applicant used generic shear modulus degradation and damping ratio curves developed by EPRI-SOG (EPRI TR-102293, 1993) instead of curves based on actual Resonant Column/Torsional Shear (RCTS) data. In FSAR Subsection 2.5S.2.5.1, the applicant refers to a comparison of the results from five site-specific RCTS tests with generic EPRI-SOG curves (EPRI TR-102293, 1993). The applicant observed a good correlation up to $10^{-2}$ percent strain. However, the applicant also observed some divergence from the selected EPRI-SOG values above the $10^{-2}$ percent strain for samples from load bearing soil layers M and N. The staff was concerned that the generic EPRI-SOG curves may not be representative of the actual strain dependent behavior of the site soils, because the applicant did not perform an adequate number of site-specific RCTS tests. Thus, the staff issued RAI 02.05.02-17, requesting the applicant to incorporate a larger number of site-specific RCTS tests into the site-response calculations.

In its response to RAI 02.05.02-17, dated July 2, 2008 (ML081890239), the applicant included a FSAR Commitment (COM 2.5S-1) that described the results of 16 RCTS tests. The applicant performed these tests on undisturbed samples from depths of 3 m (10 ft) to180 m (590 ft). The applicant then selected appropriate shear modulus degradation and damping curves published in the literature (e.g., EPRI-SOG [EPRI TR-102293, 1993] and Vucetic and Dobry [1991]) and based on comparisons with the RCTS data. The applicant then performed new site-response calculations using these curves. The applicant also revised FSAR Section 2.5.4, which included a description of the RCTS test results and the applicant's basis for selecting published shear modulus degradation and damping curves to represent the RCTS data. In the revised FSAR, the applicant also provides figures (i.e., FSAR Figures 2.5S.4-62 through 2.5S.4-68) comparing the RCTS test results with the EPRI (1993) and Vucetic and Dobry (1991) shear modulus degradation and damping curves. After reviewing the applicant's response to RAI 02.05.02-17, including Commitment (COM 2.5S-1) and the revised FSAR sections, the staff concluded that the curves used by the applicant match the data from the 16 RCTS tests and therefore, the staff concludes that the applicant has accurately characterized the subsurface soil dynamic properties at the site. Therefore, RAI 02.05.02-17 is resolved and closed.

In FSAR Subsection 2.5S.2.5, the applicant stated that an S-wave velocity of 2.8 km (9,200 ft/s) is located at a depth of more than 9,144 m (30,000 ft) below the ground surface. However, FSAR Figure 2.5S.4-57, "Deep Shear Wave Velocity Profile for the STP Site," which plots the deep S-wave velocity profile for the STP, Units 3 and 4, site, indicates that below 2,500 ft, the S-wave velocity is approximately 2.8 km (9,200 ft/s). Additionally, in Commitment (COM 2.5S-1), the applicant indicates that "below 2,500 ft depth, a hard rock shear wave velocity of 2,830 m/s (9,285 ft/s) was assumed." The staff issued RAI 02.05.02-25, requesting the applicant to clarify this discrepancy.

In its response to RAI 02.05.02-25, dated September 21, 2009 (ML092710096), the applicant indicated that the FSAR correctly states that an S-wave velocity of 2.8 km/s (9,200 ft/s) exists at a depth of more than 9,144 m (30,000 ft) below the ground surface. However, the applicant stated that for the purpose of the site-response calculations, the soil column profile is truncated at a depth of 2,469 m (8,100 ft) and below this depth, bedrock is assumed to have an S-wave velocity of 2.8 km/s (9,200 ft/s). The applicant noted that this soil column truncation depth was selected in order to capture the seismic response for frequencies greater than or equal to 0.1 Hz. The applicant will replace FSAR Figure 2.5S.4-57 with a new figure showing the S-wave velocity profiles derived from deep sonic log data, which were obtained from existing oil wells in the STP site vicinity. The data from the deep sonic log shows that at the 762-m

(2,500-ft) depth, the average S-wave velocity is approximately 3,000 fps. With respect to the statement in Commitment (COM 2.5S-1), the applicant originally based S-wave velocities below a depth of 182 m (600 ft) on a generic Mississippi embayment lowlands profile (i.e., an S-Wave velocity of 2.83 km/s (9,285 ft/s) defined below a depth of 762 m (2,500 ft). The applicant subsequently modified the above approach and used the updated S-wave velocity profile in the analysis, which is consistent with the soil profile description in Revision 3 to FSAR Subsection 2.5S.2.5. However, in reviewing the relevant contents in Subsection 2.5S 4.7.2.2.1, the staff found that Figure 2.5.S.4-57 and the corresponding Table 2.5.4-28, as well as the contents in the subsection, still indicate an S-wave velocity of 2.8 km/s (9,200 ft/s) at the depth of 762 m (2,500 ft). To address this inconsistency, the applicant revised FSAR Section 2.5S.4 to reflect the appropriate STP site-specific S-wave velocity profile. The staff confirmed that the proposed change to reflect the appropriate STP site-specific S-wave velocity profile was included in COL FSAR Revision 4. Therefore, this issue in RAI 02.05.02-25 is resolved and closed.

In summary, the staff reviewed that applicant's response to RAI 02.05.02-25, and concluded that the applicant has adequately clarified the discrepancy between FSAR Subsection 2.5S.2.5, FSAR Figure 2.5S.4-57, and Commitment (COM 2.5.S-1). The staff also concluded that the applicant's use of this new S-wave velocity profile, which is based on deep sonic log data rather than the more generic Mississippi embayment lowlands profile, is acceptable because it is based on actual data collected from the STP site vicinity.

Another important site property is kappa ($\kappa$), which estimates the dissipation of seismic energy beneath the site during an earthquake due to damping within soil layers and waveform scattering at layer boundaries. As summarized in SER Subsection 2.5S.2.2.5, the applicant uses estimates of kappa to determine an appropriate damping ratio value for the soil layers below a depth of 182 m (600 ft). The applicant assumed that these deeper soil layers behave linearly during earthquake shaking (i.e., characterized by a constant damping ratio).

As noted above in the response to RAI 02.05.02-25, Commitment (COM 2.5 S-1) states that the applicant replaced the site-specific S-wave velocity profile below the depth of 182 m (600 ft) with a new S-wave velocity profile based on deep sonic log data. In RAI 02.05.02-26, the staff asked the applicant to describe the corresponding changes to kappa as a result of this revised S-wave velocity profile. In its response to RAI 02.05.02-26, dated September 21, 2009 (ML092710096), the applicant indicated that using a new S-wave velocity profile does not affect the kappa estimate for the soils below a depth of 182 m (600 ft). The applicant subtracted the $\kappa$ value calculated for the upper 182 m (600 ft) from the total or base case $\kappa$ value (0.040 s) to obtain a residual $\kappa$ value for the soil layers below 182 m (600 ft). The applicant then used the new S-wave velocity profile and the residual $\kappa$ value to calculate a damping ratio of 0.6 percent for these deeper soils. The staff concluded that the applicant's response to RAI 02.05.02-26 is acceptable, because it considered the new S-wave velocity profile in the calculation of the damping ratio for soil layers below a depth of 182 m (600 ft). Therefore, RAI 02.05.02-26 is resolved and closed.

**Site Response Methodology**

In FSAR Subsection 2.5S.2.5.4, the applicant stated that it used RVT to calculate the site response. However, the staff concluded that the applicant did not provide sufficient detail regarding the implementation of the RVT approach. The staff issued RAI 02.05.02-18, requesting that the applicant provide a step-by-step description of how the RVT method was used to calculate soil responses at the STP site, including input parameters and modeling assumptions. In its response to RAI 02.05.02-18, dated October 1, 2008 (ML082770138), the applicant described using the Bechtel computer program SHAKE (P-SHAKE), which implements RVT, to calculate the site response. The applicant then summarized the major steps in P-SHAKE and also provides a technical paper by Deng and Ostadan (2008), which described the RVT approach and the implementation of this approach in P-SHAKE.

Deng and Ostadan (2008) state that not requiring time histories as input is a main advantage of RVT. Instead, a target response spectrum can be used directly as input. In comparison, the widely used SHAKE computer program (Idriss and Sun, 1992; Schnabel et al., 1972) typically requires a suite of time histories as input that are usually generated by matching recorded earthquake time histories to a rock motion target response spectrum, which was obtained from the seismic hazard analysis. A disadvantage is that using several time histories, in spite of all matching the same target spectrum, results in a range of amplified ground motions.

As mentioned above, an advantage of the RVT method is that a target response spectrum can be used directly as input. According to Deng and Ostadan (2008), the input target rock response spectrum is then converted to a power spectral density (PSD) function.

Next, the PSD of responses in the soil column are computed based on the input PSD and the transfer functions of the site. The statistical means of the maximum shear strains and effective shear strains are obtained based on the PSD and the process is repeated until the strain compatibility is reached over the entire soil column. Finally, the PSDs and the statistical means of the maximum responses of other required quantities, such as the acceleration response spectra and maximum accelerations, are computed once convergence on soil properties has been reached.

Deng and Ostadan (2008) also present the results of a numerical example that compared the results from P-SHAKE (Bechtel, 2006) with SHAKE. Deng and Ostadan (2008) concluded that the results show very good to excellent agreement between the two solutions.

The staff reviewed the applicant’s response to RAI 02.05.02-18 and concluded that the applicant has sufficiently described the implementation of the RVT approach in the computer program P-SHAKE. Because RG 1.208 endorses the RVT methodology, and the numerical comparison in Deng and Ostadan (2008) demonstrated that P-SHAKE is able to achieve results similar to the widely used SHAKE program, RAI 02.05.02-18 is closed.

As summarized above, the applicant’s response to RAI 02.05.02-18 attached a technical paper by Deng and Ostadan (2008) that detailed the RVT methodology and presented a numerical comparison between the P-SHAKE and SHAKE programs. The staff reviewed Deng and Ostadan (2008) and determined that the soil profile description used in the numerical comparison did not match the STP, Unit 3 and 4, site soil profile description in FSAR Section 2.5S.4. Thus, the staff issued RAI 02.05.02-23, requesting the applicant to explain the soil profile discrepancy and provide site-specific soil property data, in order to facilitate the staff’s confirmatory analysis. In addition, the staff requested the applicant to provide more details regarding the RVT methodology in the FSAR.

In its response to RAI 02.05.02-23, dated September 21, 2009 (ML092710096), the applicant indicated that the soil profile included in the Deng and Ostadan (2008) technical paper is a generic soil profile used for its numerical comparison of the P-SHAKE and the SHAKE programs. The applicant also commited to revise the FSAR with a more detailed discussion of the RVT approach and includes these proposed revisions to FSAR Subsection 2.5S.2.5.4 in the response to RAI 2.5.2-23. In addition, the applicant provides the 60 site-specific randomized soil profiles used in the site-response analysis.

The staff reviewed the applicant's response to RAI 02.05.02-23, and concluded that the applicant's use of a generic soil profile, rather than the site-specific profile, is appropriate to demonstrate the adequacy of the RVT approach for the site-response analysis. The staff also reviewed the applicant's revised FSAR sections and concluded that these revisions contain the appropriate level of detail for the FSAR. The staff found that these revisions are consistent with the applicant's description of the RVT methodology provided in the response to RAI 02.05.02-18.

Furthermore, the staff concluded that the applicant has provided all of the relevant site-specific soil property data and sufficient information to address the staff's questions. Therefore, RAI 02.05.02-23 is resolved and closed.

**NRC Site Response Confirmatory Analysis**

To determine the adequacy of the applicant's site-response calculations, the staff performed an independent confirmatory site-response analysis. As input to these calculations, the staff used the static and dynamic soil properties described in FSAR Section 2.5S.4. In addition, the staff used both the low- and high-frequency 10-4 and 10-5 rock spectra included in FSAR Tables 2.5S.2-18, "Horizontal 10-4 Rock and Site Specific UHRS (in g)," and 2.5S.2-19, "Horizontal 10-5 Rock and Site Specific UHRS (in g)," to represent the base rock input motions. The staff also used the strong motion duration values provided in FSAR Table 2.5S.2-20, "Table 2.5S.2-20 Input Rock Motion Durations," as well as applicant's selected effective strain ratio of 0.65. To be consistent with the applicant's methodology, the staff performed site-response calculations using the RVT approach. The staff calculated amplification factors that were higher than the applicant's for both the $10^{-4}$ (high-frequency) and $10^{-5}$ (both high- and low-frequency) hazard levels. For example, at 10 Hz, the staff's low-frequency $10^{-5}$ amplification is a factor of 1.5 times higher than that of the applicant. At 0.6 Hz, the staff's low-frequency $10^{-5}$ amplification is a factor of 1.25 times higher than that of the applicant. The source of the discrepancy between the staff's and applicant's site response results was due to an error in the staff's RVT software. After the software developer corrected the error, the staff's amplification factors were in close agreement with the applicant's results.

2.5S.2.4.6 Ground Motion Response Spectra

As stated in SER Section 2.5S.2, RG 1.208 defines the GMRS as the site-specific SSE to distinguish it from the CSDRS, the design ground motion for the General Electric (GE) ABWR certified design.

FSAR Subsection 2.5S.2.6, describes the method used by the applicant to develop the horizontal and vertical site-specific GRMS. To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and in ASCE/SEI Standard 43-05. To develop the vertical GMRS, the applicant also used a performance-based approach after applying V/H ratios based on NUREG/CR–6728 to the horizontal $10^{-4}$ and $10^{-5}$ soil UHRS. The applicant's horizontal and vertical GMRS are depicted in SER Figures 2.5S.2-4 and 2.5S.2-5, respectively.

Because the applicant used the standard procedure outlined in RG 1.208, to develop both the horizontal and vertical GMRS, the staff concluded that the applicant's GMRS adequately represent the STP, Units 3 and 4, site ground motion.

2.5S.2.4.7 Staff Conclusions Regarding Vibratory Ground Motion

The staff found that the applicant has adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA, and that this PSHA follows the guidance in RG 1.208. The staff also concluded that the controlling earthquakes and associated ground motion derived from the applicant's PSHA are consistent with the seismogenic region surrounding the STP site. In addition, the staff concluded the site specific GMRS, which was developed using the performance-based approach, adequately represents the regional and local seismic hazards and site effects.

### *2.5S.2.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.5S.2.6 Conclusion*

The staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information relating to vibratory ground motion, thus resolving COL License Information Item 2.24. The staff found that no outstanding information is expected to be addressed in the STP COL FSAR related to this subsection.

As stated above, the staff reviewed the seismic information submitted by the applicant in STP COL FSAR Section 2.5S.2. The staff reviewed FSAR Section 2.5S.2 and found that the applicant has provided a thorough characterization of the seismic sources surrounding the STP site, as required by 10 CFR 100.23. In addition, the staff finds that the applicant has adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA that follows the guidance in RG 1.208. Furthermore, the applicant's GMRS, which was developed using the performance-based approach, adequately represents the regional and local seismic hazards and accurately includes the effects of the local site subsurface properties. Therefore, the staff finds that the proposed COL site is acceptable from a geologic and seismologic standpoint and meets the requirements of 10 CFR 100.23.

## 2.5S.3 Surface Faulting

### *2.5S.3.1 Introduction*

This section of the FSAR addresses the potential for surface deformation due to faulting. The applicant collects information related to this category of surface deformation during site characterization investigations. The applicant's geologic, seismic, and geophysical information addresses the following specific topics related to surface faulting: geologic evidence (or the absence of evidence) for tectonic and non-tectonic surface deformation; the correlation between earthquakes with capable tectonic sources and the characterization of those sources; the ages of the most recent geologic deformation; relationships between tectonic structures in the site area and regional tectonic structures; the designation of zones of Quaternary (less than 1.8 million years ago, or 1.8 Ma) deformation in the site region; and the potential for surface deformation at the site.

### *2.5S.3.2 Summary of Application*

In Section 2.5S.3 of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant provides site-specific supplemental information COL License Information Item 2.25 identified in DCD Tier 2, Revision 4, Section 2.3. This section contains an evaluation of the potential for tectonic surface deformation and non-tectonic surface deformation at the STP, Units 3 and 4, site.

<u>*COL License Information Item*</u>

- COL License Information Item 2.25 Surface Faulting

This item evaluates site-specific geologic data to ensure that no potential exists for surface faulting at the site.

The applicant developed FSAR Section 2.5S.3, after reviewing the relevant published geologic literature; conducting geologic field investigations; and interviewing experts in geology, seismology, and tectonics of the site region. The applicant's field investigations include geologic field and aerial reconnaissance, subsurface geophysical and geotechnical investigations, and aerial photographic and remote sensing imagery analyses. In addition, the applicant uses the previous UFSAR (STPEGS, 2006) for the existing STP, Units 1 and 2, to supplement recent geologic investigations of the site.

The applicant concluded in FSAR Section 2.5S.3, that no capable tectonic faults exist in the STP vicinity or within a 40-km (25-mile) radius of the site. Additionally, the applicant concluded that there are no growth faults whose surface projections lie within a 0.6-km (1-mile) radius of the STP site. Therefore, there is a negligible potential for growth fault-induced surface deformation at the STP site location or within the STP, Units 3 and 4, footprint. The applicant applied the information in FSAR Section 2.5S.3, toward developing a basis for evaluating the geologic and seismic hazards discussed in previous and succeeding sections of the FSAR. After reviewing the data, the applicant presents the following information related to surface faulting at the STP COL site.

2.5S.3.2.1 Geologic, Seismic, and Geophysical Investigations

FSAR Subsection 2.5S.3.1, describes the information that the applicant used to evaluate the potential for surface deformation at the STP site, including: (1) previous site investigations for STP, Units 1 and 2; (2) geologic maps and data published by the USGS and the State of Texas; (3) additional published data and literature, especially information that postdates the UFSAR for STP, Units 1 and 2, and the 1986 EPRI seismic source model studies; (4) seismicity data collected before and since the 1986 EPRI studies; (5) interpretations of aerial and remote sensing imagery; and (6) results from field and aerial reconnaissance investigations. In FSAR Subsection 2.5S.3.1, the applicant stated that no data published since the UFSAR for STP, Units 1 and 2, or since the 1986 EPRI studies contradict the conclusions in the UFSAR for STP, Units 1 and 2. Based on this information, the applicant concluded that there is no evidence for Quaternary age faulting in the STP site area (within an 8-km (5-mile) radius of the site).

2.5S.3.2.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

FSAR Subsection 2.5S.3.2, discusses the geologic evidence (or the absence of evidence) for tectonic and non-tectonic surface deformation in the STP site area. The applicant concluded that there are no mapped faults in the STP site area that originate or extend into the crystalline basement rock. The applicant described growth fault studies conducted for the existing STP, Units 1 and 2, as well as recent investigations conducted for the STP, Units 3 and 4, COL application. The applicant discussed these previous and recent investigations in more detail in FSAR Subsection 2.5S.1.2.4.2. The applicant described the results of recent investigations with respect to growth fault “I” (Matagorda STP 12I) that were previously documented in the UFSAR for STP, Units 1 and 2. The surface projection of growth fault “I” approaches the STP site within the 8 km (5 mile) site area radius. The applicant stated that growth fault “I” is characterized by subtle monoclinal flexure recognizable in aerial photographs and during aerial reconnaissance investigations. Linear topographic breaks associated with this growth fault are also evident from these investigations. The applicant concluded that there is no evidence to suggest that this fault extends into the STP cooling reservoir less than about 6 km (4 miles) from the STP, Units 3 and ,4 footprint. The applicant also concluded that there is no potential for permanent ground deformation due to activity on growth fault “I” within the 1-km (0.6-mile) site radius.

2.5S.3.2.3 Correlation of Earthquakes with Capable Tectonic Sources

In FSAR Subsection 2.5S.3.3, the applicant concluded that there is no record of seismicity associated with earthquakes that have an $m_b$ greater than 3.0 within the STP site vicinity. Therefore, no spatial correlation is evident between earthquake seismicity and geologic structures within the 40-km (25-mile) site radius.

2.5S.3.2.4 Ages of Most Recent Deformations

In FSAR Subsection 2.5S.3.4, the applicant concluded that the most recent tectonic deformation of the crystalline basement rock (within the STP site vicinity) occurred during the Mesozoic Period (248 to 65 Ma). The applicant described the results from previous growth fault investigations and concluded that based on these studies, the most recent movement on a growth fault in the STP site area likely occurred more than 100,000 years ago.

2.5S.3.2.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Sources

In FSAR Subsection 2.5S.3.5, the applicant concluded that no mapped tectonic bedrock faults exist within the STP site area. Therefore, no correlation between mapped faults and regional tectonic structures is evident. Growth faults exist in the STP site area. However, growth faults are not considered capable tectonic sources because they do not penetrate the crystalline basement rock. Therefore, they are not likely to produce significant earthquakes with strong vibratory ground motions.

2.5S.3.2.6 Characterization of Capable Tectonic Sources

In FSAR Subsection 2.5S.3.6, the applicant concluded that no capable tectonic structures exist within the STP site area.

2.5S.3.2.7 Designation of Zones of Quaternary Deformation in the Site Region

In FSAR Subsection 2.5S.3.7, the applicant stated that no zones of Quaternary tectonic deformation exist in the site area. The applicant noted that there is evidence that the surface projection of one growth fault, growth fault “I,” approaches within 6 km (3.8 miles) of STP, Units 3 and 4. However, based on this distance from the STP, Units 3 and 4, footprint, the applicant did not conduct further investigations of growth fault “I,” other than the investigations discussed in FSAR Subsection 2.5S.1.2.4.2.2.2 and in FSAR Subsection 2.5S.3.1.

#### 2.5S.3.2.8 Potential for Surface Tectonic Deformation at the Site

In FSAR Subsection 2.5S.3.8, the applicant stated that no capable tectonic faults exist in the STP site vicinity and concludes that there is a negligible potential for tectonic deformation at the STP site. The applicant discussed the potential for non-tectonic surface deformation at the site, including deformation due to growth faulting, and concludes that this potential is also negligible. In addition, the applicant discusses the potential for non-tectonic surface deformation due to the following processes: (1) glacially-induced faulting, (2) collapse structures due to dissolution, (3) deformation due to salt migration at depth, (4) faulting due to volcanic activity, (5) surface collapse due to mining or oil and gas extraction, and (6) subsidence due to shallow aquifer dewatering or petroleum resource removal. The applicant concluded that with the exception of dewatering at the site, these other sources of non-tectonic deformation are not factors at the STP site. The applicant stated that subsidence due to dewatering could reach maximum levels of 1.2 to 1.5 m (0.4 to 0.5 ft). However, this occurrence is unlikely due to the ability of other sources, such as storm water, to refill the shallow aquifers.

### *2.5S.3.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the surface faulting, and the associated acceptance criteria, are in Section 2.5.3 of NUREG–0800. The acceptance criteria for reviewing COL License Information Item 2.25 are in Section 2.5.3 of NUREG–0800.

In particular, the applicable regulatory requirements for reviewing the applicant's discussion of surface faulting are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying geologic 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 a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 100.23, "Geologic and seismic siting criteria," as it relates to determining the potential for surface tectonic and non-tectonic deformations at and in the region surrounding the site.

The related acceptance criteria from Section 2.5.3 of NUREG–0800 are as follows:

- Geologic, Seismic, and Geophysical Investigations: Requirements of 10 CFR 100.23, are met and guidance in RGs 1.132, 1.198, 1.208, and 4.7 is followed for this area of review if discussions of Quaternary tectonics, structural geology, stratigraphy, geochronologic methods used for age dating, paleoseismology, and geologic history of the site vicinity, site area, and site location are complete, compare well with studies conducted by others in the same area, and are supported by detailed investigations performed by the applicant.

- Geologic Evidence, or Absence of Evidence, for Surface Tectonic Deformation: Requirements of 10 CFR 100.23, are met and guidance in RGs 1.132, 1.198, 1.208, and 4.7 is followed for this area of review if sufficient surface and subsurface information is provided by the applicant for the site vicinity, site area, and site location to confirm the presence or absence of surface tectonic deformation (i.e., faulting) and, if present, to demonstrate the age of most recent fault displacement and ages of previous displacements.

- Correlation of Earthquakes with Capable Tectonic Sources: Requirements of 10 CFR 100.23, are met for this area of review if all reported historical earthquakes within the site vicinity are evaluated with respect to accuracy of hypocenter location and source of origin, and if all capable tectonic sources that could, based on fault orientation and length, extend into the site area or site location are evaluated with respect to the potential for causing surface deformation.

- Ages of Most Recent Deformation: Requirements of 10 CFR 100.23, are met for this area of review if every significant surface fault and feature associated with a blind fault (any part of which lies within the site area) is investigated in sufficient detail to demonstrate, or allow relatively accurate estimates of, the age of most recent fault displacement and to enable the identification of geologic evidence for previous displacements (if such evidence exists).

- Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures: Requirements of 10 CFR 100.23, are satisfied for this area of review by a discussion of structural and genetic relationships between site area faulting or other tectonic deformation and the regional tectonic framework.

- Characterization of Capable Tectonic Sources: Requirements of 10 CFR 100.23, are met for this area of review when it has been demonstrated that investigative techniques employed by the applicant are sufficiently sensitive to identify all potential capable tectonic sources within the site area, such as faults or structures associated with blind faults; and when fault geometry, length, sense of movement, amount of total displacement and displacement per faulting event, age of latest and any previous displacements, recurrence rate, and limits of the fault zone are provided for each capable tectonic source.

- Designation of Zones of Quaternary Deformation in the Site Region: Requirements of 10 CFR 100.23, regarding the designation of zones of Quaternary deformation in the site region are met if the zone (or zones) designated by the applicant as requiring detailed faulting investigations is of a sufficient length and width to include all Quaternary deformation features potentially significant to the site, as described in RG 1.208.

- Potential for Surface Tectonic Deformation at the Site Location: To meet requirements of 10 CFR 100.23, for this area of review, information must be presented by the applicant in this subsection if field investigations reveal that surface or near-surface tectonic deformation along a known capable tectonic structure (i.e., a known capable tectonic feature related to a fault or blind fault) must be taken into account at the site location.

In addition, the geologic characteristics should be consistent with appropriate sections from RGs 1.132, RG 1.198, RG 1.206, RG 1.208, and RG 4.7.

### *2.5S.3.4 Technical Evaluation*

The staff reviewed the information in Section 2.5S.3 of the STP, Units 3 and 4, COL FSAR:

<u>*COL License Information Item*</u>

- COL License Information Item 2.25 Surface Faulting

The staff reviewed the applicant's information in FSAR Section 2.5S.3. Specific information in this section includes the description and evaluation of the potential for tectonic and non-tectonic surface deformation due to faulting at the STP site.

This SER section presents the staff's evaluation of the geologic, seismic, and geophysical information submitted by the applicant in FSAR Section 2.5S.3 to address the potential for surface or near-surface deformation within a 40-km (25-mile) radius of the STP COL site (i.e., the site vicinity). The staff reviewed and evaluated the submitted information to determine whether the applicant has complied with the applicable regulations and has conducted all investigations at an appropriate level of detail, in accordance with RG 1.208.

To thoroughly evaluate the applicant's geologic, seismic, and geophysical information, the staff obtained assistance from experts at the USGS. The staff and USGS counterparts visited the COL site to confirm the applicant's interpretations, assumptions, and conclusions that relate to the potential for surface or near-surface faulting and non-tectonic deformation at the site.

The applicant concluded in FSAR Section 2.5S.3, that there are no capable faults within the STP site vicinity. In addition, the applicant does conclude that potentially active growth faults exist in the site vicinity. However, the applicant also concluded that there is a negligible potential for non-tectonic surface deformation at the STP site, due to growth faulting. The staff's review of FSAR Section 2.5S.3, is presented below.

2.5S.3.4.1 Geologic, Seismic and Geophysical Investigations for Surface Deformation

The staff reviewed the geologic, seismic, and geophysical investigations that the applicant discussed in FSAR Subsection 2.5S.3.1. The applicant compiled and reviewed existing data and literature, interpreted aerial photography and remote sensing imagery, and implemented a field and aerial reconnaissance investigation. This information formed the basis for the applicant's conclusions regarding the potential for tectonic and non-tectonic surface deformation at the STP site.

The staff issued RAI 02.05.03-1, requesting the applicant to clarify the use of seismic reflection data for evaluating small (and large) fault displacements, given the resolution of the data. In its response to RAI 02.05.03-1, dated August 27, 2008 (ML082490086), the applicant stated that the seismic reflection data (as discussed with respect to previous site investigations), were only used to rule out potential surface deformation due to aseismic slip on growth faults. The staff concluded that the applicant has appropriately clarified the use of seismic reflection data for previous site investigations. Therefore, RAI 02.05.03-1 is resolved and closed.

Based on a review of FSAR Subsection 2.5S.3.1, verifications made during the staff's site visit, and a review of recent literature, the staff concluded that the applicant has performed adequate investigations to evaluate the potential for surface deformation in the STP site area, as required by 10 CFR 100.23. The following SER sections document how the applicant has implemented these investigations.

2.5S.3.4.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

The staff's review of FSAR Subsection 2.5S.3.2, focused on the applicant's description of growth faults within the site area and site vicinity. The staff concluded that the applicant's evaluation of surface faulting is adequate based on the fact that it is consistent with the existing literature. The applicant concluded that there is no evidence for surface displacement above the location where faults "A" and "I" project to the surface, and no evidence for any additional faults that might project to the surface within the STP site location. The applicant conducted new air photo analyses and field and aerial investigations to search for evidence of surface deformation associated with growth faults. The applicant concluded that there is monoclinal flexure of the ground surface associated with growth fault "I." This surface folding is documented further in FSAR Subsection 2.5S.1.2.4, and was evaluated by the staff in Subsection 2.5S.1.4.2, of this SER.

2.5S.3.4.3 Correlation of Earthquakes with Capable Tectonic Sources

The staff reviewed FSAR Subsection 2.5S.3.3, including the applicant's evaluation of seismicity data for the STP site. The applicant concluded that there is no seismicity that can be correlated with tectonic structures in the site vicinity. The staff noted that a majority of the southern portion of the site vicinity is covered by the Gulf of Mexico. The staff issued RAI 02.05.03-3, requesting the applicant to discuss the seismic potential in the Gulf region as a consequence of capable faults that may be concealed by the Gulf waters. In its response to RAI 02.05.03-3, dated September 4, 2008 (ML0825530449), the applicant acknowledged that two earthquakes occurred in the Gulf of Mexico in 2006, and that these two earthquakes motivated the applicant to revise the maximum magnitude distributions for some GCSZ sources as part of its PSHA. The applicant further discusses the revised maximum magnitudes and the PSHA for the site in FSAR Section 2.5S.2. The applicant stated that neither of the two recent Gulf of Mexico earthquakes has been linked to any tectonic source, and there are no known capable tectonic structures in the offshore portion of the STP site region. The applicant concluded that even though specific faults have not been identified to account for potential offshore seismicity, the potential for earthquakes to occur in this area is taken into account in the applicant's PSHA. Furthermore, the applicant included revised maximum magnitudes in its PSHA to reflect the largest known magnitude earthquakes that have occurred in the GCSZ.

After reviewing the applicant's response to RAI 02.05.03-3, the staff acknowledged that there is currently no evidence for capable tectonic structures in the offshore portion of the site region, and that the bedrock is concealed by tens of meters of unconsolidated sediments. However, the recent 2006, earthquakes demonstrate that there are seismogenic faults in the Gulf of Mexico capable of producing greater than M 5.0 earthquakes, which affect the seismic source modeling of the STP site. The staff's evaluation of the applicant's Gulf of Mexico seismic source characterization is included in Subsection 2.5S.2.4.2 of this SER. RAI 02.05.03-3 is closed.

The staff issued RAI 02.05.03-2, requesting the applicant to discuss any potential effects of migrating seismicity at the STP site. In its response to RAI 02.05.03-2, dated September 4, 2008 (ML082530449), the applicant explained that in areas like the NMSZ, numerous authors (Tuttle et al., 2006; Nelson et. al., 1999; Schweig and Ellis, 1994; and Coppersmith, 1988) have speculated that large earthquakes may occur at different locations, along different faults, and at different periods in time (i.e. migrating seismicity). The applicant stated that the effects of migrating seismicity are not a factor at the site for the following reasons: (1) the tectonic setting of the STP site is different from that of the NMSZ; (2) there are no known capable structures within the STP site region that large earthquakes could migrate to; and (3) the EPRI-SOG seismic sources used in the STP seismic hazard calculations were updated based on recent geologic or seismic information. The staff reviewed the applicant's response to RAI 02.05.03-2, and concluded that the applicant has adequately evaluated the potential for migrating seismicity to affect the STP site, and there is no evidence to support migrating seismicity at the site. Therefore, RAI 02.05.03-2 is resolved and closed.

After reviewing the information in FSAR Subsection 2.5S.3.3, and in the applicant's responses to RAI 02.05.03-2 and RAI 02.05.03-3, the staff concluded that the applicant has presented convincing data and logical interpretations related to a lack of correlation between earthquakes and tectonic sources at the STP site. The applicant has adequately justified that there is no correlation between seismicity and capable tectonic structures at the STP site. This conclusion is based on information available in the existing literature and a lack of seismicity in the STP site vicinity. The staff concluded that the applicant's information in FSAR Subsection 2.5S.3.3 is in accordance with the guidance in RG 1.208 and meets the requirements of 10 CFR 100.23.

2.5S.3.4.4 Ages of Most Recent Deformation

The staff reviewed possible ages for growth faults in the STP site area. The staff's review focused on those faults that displace Tertiary and younger strata, specifically growth fault Matagorda STP12I (Matagorda GMO). The staff's evaluation of potential Quaternary deformation associated with growth fault STP 12I/GMO is included in Subsection 2.5S.1.4.2 of this SER.

2.5S.3.4.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Sources

The applicant concluded in FSAR Subsection 2.5S.3.5 that no tectonic structures exist in the STP site area. There are growth faults in the site area that are associated with the regionally identified Frio Fault Zone. However, the applicant concluded that growth faults are not considered to be tectonic structures and are not linked to any capable regional tectonic sources.

The staff concluded that the applicant has adequately evaluated the potential relationship between tectonic structures in the STP site area and regional tectonic sources. Based on existing literature, the staff concurred with the applicant that there is no evidence to suggest tectonic faulting in the STP site area. Furthermore, the staff concluded that the applicant's characterization of growth faults as non-tectonic structures is consistent with existing literature and with the guidance in RG 1.208.

2.5S.3.4.6 Characterization of Capable Tectonic Sources

The staff reviewed the applicant's conclusion that no capable tectonic sources exist in the STP site area. The applicant cites FSAR Section 2.5S.1 and FSAR Subsection 2.5S.3.4, to support this conclusion. The staff concurred with the applicant's conclusion that there is no evidence to support the presence of a capable tectonic source within the STP site area. The staff based this conclusion on: (1) a review of the applicant's information in FSAR Sections 2.5S.1 and 2.5S.3, (2) a review of the applicant's field investigations carried out within the site area and site vicinity, (3) a geologic site visit conducted by the staff and USGS advisors, and (4) a lack of identified tectonic structures in the existing literature for the STP area.

2.5S.3.4.7 Designation of Zones of Quaternary Deformation at the Site

The staff reviewed the applicant's discussion of Quaternary zones of deformation in FSAR Subsection 2.5S.3.7. The applicant concluded that only one growth fault (Matagorda STP12I/GMO) is associated with possible Quaternary deformation in the STP site area, and that fault projects 6.1 km (3.8 mi) from the proposed STP, Units 3 and 4 ,"footprint."

The staff reviewed FSAR Subsection 2.5S.3.7, and concluded that the applicant has adequately designated zones of Quaternary deformation within the STP site area.

2.5S.3.4.8 Potential for Surface Tectonic Deformation at the Site

The applicant concluded in FSAR Subsection 2.5S.3.8, that the potential for tectonic and non-tectonic surface deformation at the site is negligible. The staff reviewed the applicant's descriptions of the potential sources for surface tectonic deformation at the STP site. The staff concluded that the information in FSAR Section 2.5S.1 and FSAR Section 2.5S.3, presents no evidence for tectonically related deformation at the STP site. SER Subsection 2.5S.3.4.2 includes a review of the potential for non-tectonic surface deformation at the site.

In addition to the RAIs discussed above, the staff issued RAI 02.05.03.4, requesting the applicant to correct numerous cross references to FSAR Section 2.5S.1. In its response to RAI 02.05.03-4, dated June 26, 2008 (ML081970231), the applicant included the necessary corrections and also updates the references in the latest revision of the FSAR. Therefore, RAI 02.05.03-4 is resolved and closed.

### *2.5S.3.5 Post Combined License Activities*

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

### *2.5S.3.6 Conclusion*

The staff reviewed the geologic information in FSAR Section 2.5S.3, and considered the information the applicant has gathered during the regional and site-specific geologic, seismic, and geophysical investigations. As a result of this review, the staff finds that the applicant has performed the investigations in accordance with 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii) by following the guidance in RG 1.208. The staff finds that the applicant has provided an adequate basis for establishing that there are no known capable tectonic sources in the site vicinity that would cause surface or near-surface deformation in the site area. The staff further concluded that the site is suitable from the perspective of tectonic surface deformation and meets the requirements of 10 CFR 100.23 and 10 CFR 52.79. This finding also addresses COL License Information Item 2.25. In conclusion, the applicant has provided sufficient information to satisfy 10 CFR 100.23.

## 2.5S.4 Stability of Subsurface Materials and Foundations

### *2.5S.4.1 Introduction*

This section of the FSAR presents the applicant's evaluation of the stability of subsurface materials and foundations that relate to the STP site. The properties and stability of the soil and rock underlying the site are important to the safe design and siting of the plant. The information in this section addresses: (1) geologic features in the site vicinity; (2) static and dynamic engineering properties of soil and rock strata underlying the site; (3) the relationship of the foundations for safety-related facilities and the engineering properties of underlying materials; (4) results of seismic refraction and reflection surveys, including in-hole and cross-hole explorations; (5) safety-related excavation and backfill plans and engineered earthwork analysis and criteria; (6) groundwater conditions and piezometric pressure in all critical strata as they affect the loading and stability of foundation materials; (7) responses of site soils or rocks to dynamic loading; (8) liquefaction potential and consequences of liquefaction of all subsurface soils, including the settlement of foundations; (9) earthquake design bases; (10) results of investigations and analyses conducted to determine foundation material stability, deformation, and settlement under static conditions; (11) criteria, references, and design methods used in static and seismic analyses of foundation materials; (12) techniques and specifications to improve subsurface conditions, which are to be used at the site to provide suitable foundation conditions; and any additional information deemed necessary in accordance with 10 CFR Part 52.

### *2.5S.4.2 Summary of Application*

In Section 2.5S.4 of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant provided site-specific supplemental information to address COL License Information Items 2.26, 2.28, 2.29, 2.30, 2.31,2.32, 2.33, 2.34, 2.35, 2.36, 2.37, 2.38, and 2.39.

<u>*COL License Information Items*</u>

- COL License Information Item 2.26 Stability of Subsurface Material and Foundation

The applicant provided supplemental information to resolve COL License Information Item 2.26. COL License Information Item 2.26, addresses the properties and stability of site-specific soils and rocks under both static and dynamic conditions, including the vibratory ground motions associated with the site-specific SSE.

- COL License Information Item 2.28 Field Investigations

The applicant provided supplemental information to resolve COL License Information Item 2.28. COL License Information Item 2.28, addresses the type, quantity, extent, and purpose of all field explorations, including logs of all borings and test pits; results of geophysical surveys in tables and profiles; and records of field plate load tests, field permeability tests, and other special field tests.

- COL License Information Item 2.29 Laboratory Investigations

The applicant provided supplemental information to resolve COL License Information Item 2.29. COL License Information Item 2.29, addresses the number and type of laboratory tests and the location of samples in tabular form, including results of laboratory tests on disturbed and undisturbed soil and rock samples.

- COL License Information Item 2.30 Subsurface Conditions

The applicant provided supplemental information to resolve COL License Information Item 2.30. COL License Information Item 2.30, addresses the investigation of subsurface conditions and engineering classifications and descriptions of soil and rock supporting the foundations, including the history of soil deposition and erosion, past and present ground water levels, glacial or other preloading influences, rock weathering, and any rock or soil characteristics that may present a hazard to plant safety.

- COL License Information Item 2.31 Evacuation and Backfilling for Foundation Construction

The applicant provided supplemental information to resolve COL License Information Item 2.31. COL License Information Item 2.31, addresses the site-specific thickness and properties of soil (if any) between the base of the foundation and the underlying rock, including the configuration and detailed longitudinal sections and cross sections of other safety-related structures of the plant; the extent of all Seismic Category I excavations, fills, and slopes; the excavation and dewatering methods, and the sources, quantities, and static and dynamic engineering properties of borrowed materials and fill properties.

- COL License Information Item 2.32 Ground Water Conditions

The applicant provided supplemental information to resolve COL License Information Item 2.32. COL License Information Item 2.32, addresses the site-specific ground water conditions.

- COL License Information Item 2.33 Liquefaction Potential

The applicant provided supplemental information to resolve COL License Information Item 2.33. COL License Information Item 2.33, verifies that at site-specific SSE ground motion, no liquefaction potential exists for soils under and around all Seismic Category I structures, including Seismic Category I buried pipelines and electrical ducts through the liquefaction potential evaluation; the magnitude and duration of the earthquake; and the number of cycles of earthquakes.

- COL License Information Item 2.34 Response of Soil and Rock to Dynamic Loading

The applicant provided supplemental information to resolve COL License Information Item 2.34. COL License Information Item 2.34, determines the dynamic soil properties of the site in terms of shear modulus and material damping, as functions of shear strain used to determine the site-specific SSE ground motion.

- COL License Information Item 2.35 Minimum Static Bearing Capacity

The applicant provided supplemental information to resolve COL License Information Item 2.35. COL License Information Item 2.35, verifies that the site has the minimum static bearing capacity of 718.20 kilopascals (kPa) (104.2 pounds per square inch [psi]) at the foundation level of the reactor and control buildings, and the foundation material has adequate bearing capacity to withstand the site-specific loads.

- COL License Information Item 2.36 Earth Pressures

The applicant provided supplemental information to resolve COL License Information Item 2.36. COL License Information Item 2.36, addresses a site-specific evaluation of static and dynamic lateral earth pressures and hydrostatic ground water pressures acting on plant safety-related facilities.

- COL License Information Item 2.37 Soil Properties for Seismic Analysis of Buried Pipes

The applicant provided supplemental information to resolve COL License Information Item 2.37. COL License Information Item 2.37, addresses the provision and justification of soil properties used for the seismic analysis of Seismic Category I buried pipes and electrical conduits.

- COL License Information Item 2.38 Static and Dynamic Stability of Facilities

The applicant provided supplemental information to resolve License Information Item 2.38. COL License Information Item 2.38, performs a site-specific stability evaluation of all safety-related facilities including foundation rebound, settlement, differential settlement, and bearing capacity.

- COL License Information Item 2.39 Subsurface Instrumentation

The applicant provided supplemental information to resolve COL License Information Item 2.39. COL License Information Item 2.39, describes instrumentation, if any, proposed for the surveillance of the performance of the foundations for safety-related structures, including the type, location, and purpose of each instrument and significant details of installation methods, as well as a schedule for installing and reading all instruments, interpreting the data, and the limiting values for continued safety.

FSAR Section 2.5S.4, describes the geotechnical explorations performed at the site to determine in situ soil and rock properties, to obtain samples for laboratory testing, and to determine the laboratory tests conducted to confirm the soil and rock properties and the analyses conducted to determine the acceptability of the STP, Units 3 and 4, site against the ABWR DCD site requirements. FSAR Section 2.5S.4, data are organized into 12 subsections: FSAR Subsection 2.5S.4.1, “Geologic Features”; FSAR Subsection 2.5S.4.2, “Properties of Subsurface Materials”; FSAR Subsection 2.5S.4.3, “Foundation Interfaces”; FSAR Subsection 2.5S.4.4, “Geophysical Surveys”; FSAR Subsection 2.5S.4.5, “Excavation and Backfill”; FSAR Subsection 2.5S.4.6, “Ground water Conditions”; FSAR Subsection 2.5S.4.7, “Response of Soil and Rock to Dynamic Loading”; FSAR Subsection 2.5S.4.8, “Liquefaction Potential”; FSAR Subsection 2.5S.4.9, “Earthquake Site Characteristics”; FSAR Subsection 2.5S.4.10, “Static Stability”; FSAR Subsection 2.5S.4.11, “Design Criteria”; and FSAR Subsection 2.5S.4.12, “Techniques to Improve Subsurface Conditions.”

### 2.5S.4.2.1 Description of Geologic Features

In FSAR Subsection 2.5S.4.1, the applicant refers to FSAR Subsections 2.5.1.1 and 2.5.1.2, for a detailed description of the regional geologic settings, site-specific conditions, potential geologic hazards, and tectonic features within the STP, Units 3 and 4, site.

2.5S.4.2.2 Properties of Subsurface Materials

FSAR Subsection 2.5S.4.2, describes the static and dynamic engineering properties of the STP, Units 3 and 4, site subsurface materials, including the field investigations, laboratory tests, and engineering properties the applicant determined from the subsurface materials. The applicant stated that the field and laboratory investigations for determining the engineering properties of soil materials follow the guidance of RG 1.132 and RG 1.138 Revision 2, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," respectively.

**Description of Subsurface Materials**

FSAR Subsection 2.5S.4.2.1, reviews the subsurface profile and materials and describes the underlying strata. The applicant categorized the soils underlying the STP site into 12 different soil strata based on the physical and engineering properties of the soil determined from Standard Penetration Tests (SPTs), CPTs, tests pits, geophysical downhole suspension compression and shear wave (P-S) velocity logging, field electrical resistivity testing, and observation well installation, as well as an extensive laboratory testing program. The following sections of this SER describe each soil stratum and summarize the applicant's laboratory test results for clays and sands. SER Table 2.5S.4-1 (FSAR Table 2.5S.4-16, "Summary of Average Geotechnical Engineering Parameters"), summarizes the geotechnical engineering properties for each soil stratum.

The applicant relied on the results of laboratory tests such as unconsolidated-undrained triaxial test (UU) and unconfined compression (UNC) strength tests, and field tests such as the SPT and CPT to determine the undrained shear strength of the soil. The applicant estimated the drained friction angle (Φ') for cohesionless fine-grained soils using empirical correlations with corrected STP N-values as well as CPT data. The applicant also performed laboratory triaxial strength tests (CIU-bar) or direct shear test results to obtain the friction angle. For coarse-grained soils, the applicant based the estimate of the elastic modulus on corrected STP $N_{60}$ values and small strain shear wave velocity measurements at the STP site.

<u>Strata A through E</u>. The applicant noted that Strata A through E extend from the ground surface down to a depth of about 27 m (90 ft) and are made up of clays, silts, and fine sands. The applicant planned to excavate these strata to reach the design final subgrade for the reactor buildings at an elevation (El.) of -8.36 m (-60.25 ft). The applicant plans to find the control building on Stratum C and the turbine building on structural fill above Stratum E.

<u>Strata F through N</u>. The applicant also noted that Strata F through N extend from a depth of about 27 m (90 ft) to about 182 m (600 ft) below the surface and consist of dense sand, silt, and clay strata. The applicant plans to find the reactor buildings on concrete fill just above Stratum F.

<u>Chemical Properties of Soils</u>. FSAR Subsection 2.5S.4.2.1.13, uses field electrical resistivity and laboratory chemical tests to describe the corrosion potential of the foundation soils. The applicant conducted 46 sets of chemical tests on the soils between 0.45 and 24 m (1.5 and 80 ft) in depth. The applicant also performed four arrays of electrical resistivity tests across the site. Because the chemical tests indicate moderately corrosive soils, the applicant concluded that special protection may be required if metals are to be placed against the soils. The

applicant also noted that based on laboratory sulfate content tests, there is a less than 10 percent potential for a sulfate attack on concrete.

Deep Subsurface Conditions Deeper. FSAR Subsection 2.5S.4.2.1.14, noted that as part of the subsurface investigation performed for the STP, Units 1 and 2, sites, one boring was extended to a depth of approximately 798 m (2,620 ft) below the ground surface. The applicant stated that approximately two-thirds of the sediments encountered in the boring are fine-grained, consisting mainly of clay, silty clay, silt, claystone, or siltstone, while the remaining one-third are coarse-grained, consisting mainly of silty sandy or sand.

## Table 2.5S.4-1 Summary of Average Geotechnical Engineering Parameters (FSAR Table 2.5S.4-16)

| | Strata | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A | B | C | D | E | F | H | J CLAY | J SAND | K CLAY | K SAND/ SILT | L | M | N CLAY | N SAND |
| Average Thickness m (ft) | 5.7 (19) | 2.1 (7) | 5.7 (19) | 6.4 (21) | 5.4 (18) | 4.8 (16) | 5.3 (17.5) | 18.5 (61) | 11.4 (37.5) | 5.6 (18.5) | 7.7 (25.3) | 1.5 (5) | 4.5 (15) | >69.4 (>228) | 28.4 (119) |
| USCS Symbol | CH, CL | ML, CL, SM, SC | SM, SP-SM, ML | CH, CL, ML, CL-ML | SP-SM, SM, ML, SP, SC | CH, CL, ML, CL-ML | SP-SM, SM | CH, CL, ML | SM, ML, SP-SM, CL | CL, CH | SM, ML | CH | SM | CH, CL, SC | SM, SP-SM, SC |
| Natural Moisture Content % | 24 | 24 | 23 | 26 | 21 | 24 | 19 | 23 | 22 | 23 | 21 | 29 | 19 | 25 | 22 |
| Moist Unit Weight kg/m$^3$ (pcf) | 1,986 (124) | 1,938 (121) | 1,954 (122) | 1,954 (122) | 1,970 (123) | 2,002 (125) | 2,002 (125) | 2,002 (125) | 2,002 (125) | 1,986 (124) | 2,034 (127) | 1,986 (124) | 2,034 (127) | 1,970 (123) | 2,050 (128) |
| Fines Content % | 96 | 67 | 23 | 79 | 20 | 94 | 18 | 90 | 50 | 87 | 45 | 87 | 45 | 79 | 21 |
| Liquid Limit % | 56 | - | - | 57 | - | 57 | - | 54 | - | 50 | - | 73 | - | 67 | - |
| Plasticity Index % | 40 | - | - | 40 | - | 40 | - | 35 | - | 35 | - | 50 | - | 45 | - |
| Uncorrected SPT N-value bpf | 9 | 8 | 23 | 15 | 33 | 22 | 42 | 32 | 55 | 15 | 60 | 21 | 60 | 32 | 83 |
| Corrected SPT $N_{60}$-Value bpf | 11 | 11 | 38 | 23 | 53 | 34 | 58 | 48 | 94 | 26 | 68 | 36 | 100 | 54 | 141 |
| Corrected SPT $(N_1)_{60}$-Value, bpf | - | 12 | 35 | - | 31 | - | 28 | - | 38 | - | 27 | - | 40 | - | 56 |
| Vs m/s (fps) | 175 (575) | 220 (725) | 239 (785) | 281 (925) | 329 (1,080) | 288 (945) | 327 (1,075) | 330 (1,085) | 388 (1,275) | 356 (1,170) | 417 (1,370) | 297 (975) | 355 (1,165) | 383 (1,290) | 504 (1,655) |

| | Strata | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **A** | **B** | **C** | **D** | **E** | **F** | **H** | **J CLAY** | **J SAND** | **K CLAY** | **K SAND/ SILT** | **L** | **M** | **N CLAY** | **N SAND** |
| Undrained shear strength ($S_U$) kPa (ksf) | 71 (1.5) | - | - | 143 (3.0) | - | 162 (3.4) | - | 181 (3.8) | - | 186 (3.9) | - | 186 (3.9) | - | 215 (4.5) | - |
| Drained Friction Angle (Ø') | - | 30 | 35 | 16 | 35 | 8 | 35 | 11 | 33 | 11 | 31 | - | 31 | - | 36 |
| Drained Cohesion (c') kPa (ksf) | - | - | - | 57 (1.2) | - | 95 (2.0) | - | 110 (2.3) | - | 110 (2.3) | - | - | - | - | - |
| Elastic Modulus (High Strain, Es) MPa (ksf) | 54 (1,135) | 57 (1,200) | 86 (1,810) | 116 (2,430) | 150 (3,145) | 123 (2,570) | 155 (3,240) | 198 (4,140) | 227 (4,755) | 208 (4,350) | 235 (4,915) | 185 (3,865) | 208 (4,350) | 376 (7,855) | 557 (11,645) |
| Elastic Modulus (High Strain, $E_{(985)}$) MPa (ksf) | 47 (985) | 57 (1,200) | 86 (1,810) | 89 (1,865) | 150 (3,145) | 94 (1,970) | 155 (3,240) | 152 (3,175) | 227 (4,755) | 159 (3,335) | 235 (4,915) | 141 (2,965) | 208 (4,350) | 288 (6,020) | 557 (11,645) |
| Shear Modulus (High Strain, Gs) MPa (ksf) | 17.7 (370) | 22 (465) | 33 (695) | 38 (800) | 58 (1,215) | 40 (850) | 59 (1,250) | 66 (1,380) | 87 (1,830) | 69 (1,450) | 90 (1,890) | 62 (1,300) | 80 (1,675) | 125 (2,620) | 214 (4,470) |
| Shear Modulus (Low Strain, Gmax) MPa (ksf) | 60 (1,270) | 94 (1,970) | 111 (2,335) | 155 (3,240) | 21 (455) | 166 (3,470) | 214 (4,490) | 218 (4,570) | 302 (6,310) | 252 (5,270) | 354 (7,400) | 175 (3,660) | 256 (5,350) | 304 (6,355) | 521 (10,890) |
| Poisson's Ratio (drained) ($\mu_d$) | 0.30 | 0.30 | 0.30 | 0.15 | 0.30 | 0.15 | 0.30 | 0.15 | 0.30 | 0.15 | 0.30 | 0.15 | 0.30 | 0.15 | 0.30 |
| Coefficient of Subgrade Reaction (k1) kcf | 150 | 160 | 600 | 300 | 600 | 300 | 600 | - | - | - | - | - | - | - | - |
| Earth Pressure Coefficients | | | | | | | | | | | | | | | |
| -Active (Ka) | 0.5 | 0.3 | 0.3 | 0.5 | 0.3 | 0.5 | 0.3 | - | - | - | - | - | - | - | - |

| | Strata | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **A** | **B** | **C** | **D** | **E** | **F** | **H** | **J CLAY** | **J SAND** | **K CLAY** | **K SAND/ SILT** | **L** | **M** | **N CLAY** | **N SAND** |
| -Passive (Kp) | 2.0 | 3.0 | 3.7 | 2.0 | 3.7 | 2.0 | 3.7 | - | - | - | - | - | - | - | - |
| -At-rest ($K_{0, NC}$) | 0.7 | 0.5 | 0.4 | 0.7 | 0.4 | 0.7 | 0.4 | - | - | - | - | - | - | - | - |
| -At-rest ($K_{0, OCR}$) | 1.4 | - | - | 1.0 | - | 1.0 | - | - | - | - | | | | | |
| Sliding Coefficient | 0.30 | 0.35 | 0.40 | 0.30 | 0.40 | 0.30 | 0.40 | - | - | - | - | - | - | - | - |
| Consolidation Properties | | | | | | | | | | | | | | | |
| -Compression Index (Cc) | 0.235 | - | - | 0.285 | - | 0.238 | - | 0.224 | - | 0.176 | - | 0.176 | - | 0.336 | - |
| -Recompression Index (Cr) | 0.017 | - | - | 0.026 | - | 0.028 | - | 0.038 | - | 0.017 | - | 0.017 | - | 0.050 | - |
| -Preconsolidation Pressure (Pc') kPa (ksf) | 301 (6.3) | - | - | 580 (12.3) | - | 742 (15.5) | - | 880 (18.5) | - | 870 (18.3) | - | 981 (20.5) | - | 1,771 (37) | - |
| Overconsolidation Ratio (OCR) | 7.0 | - | - | 3.3 | - | - | - | - | - | - | - | 1.3 | - | 1.3 | - |
| ft=foot, m=meter, s=second, bpf=blow per foot, fps=foot per second, kcf=kips (1,000 pounds) per cubic foot, ksf=kips per square foot, kPa=kilopascal, MPa=Megapascal, pcf=pound per cubic foot | | | | | | | | | | | | | | | |

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Field Testing Program. FSAR Subsection 2.5S.4.2.1.15, describes the Field Testing Program, including the number of borings and tests performed in support of the COL application. The applicant stated that this program conforms to the guidance in RG 1.132, and includes an audited and approved Quality Assurance Program and site-specific work procedures. SER Table 2.5S.4-2, presents the type and number of tests performed and the standards followed.

**Table 2.5S.4-2 Field Testing Summary (FSAR Table 2.5S.4-1)**

| Field Test | Industry Standard | Number Of Tests |
|---|---|---|
| Borings (B) | ASTM D 1586 (1999)<br>ASTM D 1587 (2000) | 132 |
| SPT Hammer Energy Measurements | ASTM D 6066 (2004)<br>ASTM D 4633 (2005) | 52 |
| Cone Penetration Tests (C) | ASTM D 5778 (1995) | 44 |
| Observation Wells | ASTM D 5092 (2004) | 28 |
| Test Pits (TP) | No Standard | 6 |
| Field Electrical Resistivity Arrays (ER) | ASTM G 57 (1995)<br>IEEE 81 (1983) | 4 |
| Suspension P-S Velocity Logging | ASCE Ohya (1986) | 10 |

Laboratory Testing Program. FSAR Subsection 2.5S.4.2.1.16, describes the Laboratory Testing Program for soil samples completed as part of the COL subsurface investigation. The applicant completed the laboratory testing in accordance with the guidance in RG 1.138. The applicant also performed the testing under an approved Quality Assurance Program following work procedures developed specifically for the COL and the collected soil samples. FSAR Subsection 2.5S.4.2.2, provides additional details of the Field Laboratory Program.

**Exploration**

FSAR Subsection 2.5S.4.2.2, describes the methods and equipment used to perform the site exploration, including soil borings, ground water monitoring wells, CPT soundings, surface geophysical surveys, geotechnical test pits, as well as the number and type of explorations performed for the STP investigations.

Subsurface Investigation (STP, Units 3 and 4). FSAR Subsection 2.5S.4.2.2.2, states that the applicant performed subsurface investigations at the STP, Units 3 and 4, site between October 2006, and January 2007, and again during the summer of 2008. FSAR Figures 2.5S.4-1 and 2.5S.4-2, identify the field testing locations.

Boring and Sampling. FSAR Subsection 2.5S.4.2.2.3, states that 132 borings were drilled around and outside of the power block area to a maximum depth of approximately 182 m (600 ft) for the COL site investigation. The applicant drilled 32 additional borings to depths ranging from 54.8 to 91.4 m (180 to 300 ft), which focused on the relocation of the ultimate heat sink (UHS) basins, UHS pump houses, reactor service water (RSW) tunnels, and diesel generator fuel storage vaults. The applicant collected soil samples in accordance with the American Society for Testing and Materials (ASTM) standards D 1586, 1587, 2113, and 4633, among others. The applicant collected soil samples using the SPT sampler at 0.75 m (2.5 ft) intervals to a depth of about 4.5 m (15 ft); at 1.5 m (5 ft) intervals from depths of 4.5 m (15 ft) to 30.48 m (100 ft); at 3.048 m (10 ft) intervals from depths of 30.48 m (100 ft) to 60.96 m (200 ft); and at 6.096 m (20 ft) sample intervals to a maximum depth of approximately 182 m (600 ft).

The applicant used either a Shelby tube sampler or a rotary pitcher sampler to retrieve the undisturbed samples. The applicant labeled and transported all tubes to the sample storage area and testing facilities in accordance with ASTM D 4220.

Cone Penetration Testing. The applicant conducted CPTs in accordance with ASTM D 5778 and measured tip resistance, sleeve friction, and porewater pressure. The applicant advanced each CPT to depths ranging from 10.9 to 30.48 m (36 to 100 ft) below the surface. The applicant also performed seismic testing and pore pressure dissipation tests in six and ten CPTs, respectively.

Observation Wells and Slug Testing. The applicant installed 28 observation wells ranging in depth from approximately 10.9 to 36 m (36 to 121 ft) below the surface. The applicant performed well installation in accordance with ASTM D 5092 and utilized ASTM D 4044 to perform permeability tests in each of the observation wells.

Test Pits. FSAR Subsection 2.5S.4.2.2.6, states that the applicant excavated six test pits to a maximum depth of approximately 2.74 m (9 ft) below the ground surface at the site to collect soil samples for laboratory testing.

Geophysical Logging. FSAR Subsection 2.5S.4.2.2.8, describes the applicant's geophysical testing methods in ten boreholes including suspension P-S velocity logging, natural gamma, long and short resistivity, spontaneous potential, and three arm caliper and deviation surveys. FSAR Section 2.5S.4.4, discusses the results of the suspension P-S velocity logging.

**Laboratory Testing**

FSAR Subsection 2.5S.4.2.3, describes the laboratory testing that the applicant performed on disturbed and undisturbed soil samples and bulk soil samples obtained during the subsurface investigation. The applicant performed the testing in accordance with ASTM and other applicable standards. SER Table 2.5S.4-3 (FSAR Table 2.5S.4-7, "Laboratory Testing Summary"), identifies the type, number, and industry standard followed for each type of laboratory test.

**Table 2.5S.4-3 Laboratory Testing Summary (FSAR Table 2.5S.4-7)**

| Laboratory Test | Industry Standard | Number Of Tests |
|---|---|---|
| Moisture Content | ASTM D 2216 (2005) | 534 |
| Atterberg Limits | ASTM D 4318 (2005) | 286 |
| Grain Size Analysis | ASTM D 422 (2002)<br>ASTM D 6913 (2004) | 257 |
| Specific Gravity | ASTM D 854 (2006) | 107 |
| Unit Weight | ASTM Standards | 141 |
| Unconsolidated Undrained Triaxial | ASTM D 2850 (2003) | 76 |

| Laboratory Test | Industry Standard | Number Of Tests |
|---|---|---|
| Strength | | |
| Unconfined Compressive Strength | ASTM D 2166 (2006) | 25 |
| Consolidated Undrained Triaxial Strength | ASTM D 4767 (2004) | 17 |
| Direct Shear Strength | ASTM D 3080 (2004) | 10 |
| Consolidation | ASTM D 2435 (2004) | 37 |
| Moisture-Density (Proctor Compaction) | ASTM D 1557 (2002) | 8 |
| California Bearing (CBR) | ASTM D 1883 (2005) | 4 |
| pH | ASTM D 4972 (2001) | 67 |
| Chloride Content | EPA 300.0 (1993) | 47 |
| Sulfate Content | EPA 300.0 (1993) | 47 |
| Resonant Column Torsional Shear (RCTS) | Stokoe et al. (2006) | 16 |

2.5S.4.2.3 Foundation Interfaces

FSAR Subsection 2.5S.4.3, describes the locations of site exploration points for the subsurface investigation including borings, observation wells, CPTs, electrical resistivity tests, and test pits made inside and outside of the power block area. FSAR Section 2.5S.4.5, addresses the excavation geometry for the safety-related structures, and other major facilities and cross sections of the structure foundations, with the proposed excavation and backfilling limits.

2.5S.4.2.4 Geophysical Surveys

FSAR Subsection 2.5S.4.4, describes the geophysical testing conducted for the STP site that includes geophysical surveys performed for STP, Units 1 and 2, as well as for the new STP, Units 3 and 4.

**Previous Geophysical Surveys for STP, Units 1 and 2**

The applicant used various geophysical methods as part of the subsurface investigation for existing STP, Units 1 and 2, including geophysical refraction and reflection surveys and geophysical borehole logging. FSAR Subsections 2.5S.4.4.1.1 through 2.5S.4.1.4, summarize these methods.

**Geophysical Survey for STP, Units 3 and 4**

FSAR Subsection 2.5S.4.4.2, describes the Suspension P-S Velocity Logging and seismic CPTs the applicant performed in order to determine the compressional (Vp) and shear wave (Vs) velocities of the soils underlying the site. The results from these surveys are discussed in the following paragraphs.

Suspension P-S Velocity Logging. The applicant performed P-S velocity logging tests to determine the average in situ Vs and Vp measurements of the soil column surrounding each borehole.

The applicant used this method to test up to a maximum depth of approximately 182 m (600 ft) below the ground surface. SER Table 2.5S.4-4 (FSAR Section 2.5S.4.4.2.1, “Suspension P-S Velocity Logging”), presents the minimum, maximum, and average shear wave velocity values in various soil strata from STP, Units 3 and 4.

**Table 2.5S.4-4 Minimum, Maximum, and Average Shear Velocity (Vs) for STP Units 3 and 4 and Average Vs for STP Units 1 and 2 from Suspension P-S Velocity Logging (FSAR Subsection 2.5S.4.4.2.1)**

| Stratum | Minimum Vs | Maximum Vs | Average Vs | Average Vs Units 1 And 2 |
|---|---|---|---|---|
| A | 88 (290) | 304 (1,000) | 170 (559) | 202 (663) |
| B | 121 (400) | 332 (1,090) | 219 (719) | 275 (905) |
| C | 134 (440) | 435 (1,430) | 236 (776) | 277 (910) |
| D | 164 (540) | 472 (1,550) | 285 (937) | 313 (1,030) |
| E | 219 (720) | 435 (1,430) | 326 (1,072) | 352 (1,155) |
| F | 219 (720) | 390 (1,280) | 288 (947) | 401 (1,316) |
| H | 222 (730) | 667 (2,190) | 323 (1,061) | 475 (1,560) |
| J Clay | 195 (640) | 573 (1,880) | 331 (1,089) | 366 (1,201) |
| J Sand | 219 (720) | 978 (3,210) | 388 (1,275) | 366 (1,201) |
| K Clay | 222 (730) | 502 (1,650) | 356 (1,170) | 469 (1,541) |
| K Sand/Silt | 286 (940) | 612 (2,010) | 417 (1,371) | 469 (1,541) |
| L | 228 (750) | 429 (1,410) | 298 (979) | 387 (1,271) |
| M | 243 (800) | 487 (1,600) | 355 (1,165) | 463 (1,520) |
| N Clay | 213 (700) | 774 (2,540) | 395 (1,296) | 403 (1,324) |
| N Sand | 265 (870) | 740 (2,430) | 504 (1,654) | 483 (1,585) |
| *All velocities are shown as m/s (fps). | | | | |

Seismic CPT Measurements. The applicant stated that the maximum depth tested by the seismic CPTs was approximately 28.9 m (95 ft) below the ground surface. SER Table 2.5S.4-5 (FSAR Subsection 2.5S.4.4.2.2) presents the minimum, maximum, and average shear wave velocity values obtained from seismic CPTs in various soil strata from STP, Units 3 and 4.

**Table 2.5S.4-5 Minimum, Maximum, and Average Vs for STP Units 3 and 4 from Seismic CPT Measurements (FSAR Subsection 2.5S.4.4.2.2)**

| Stratum | Minimum Vs | Maximum Vs | Average Vs |
|---|---|---|---|
| A | 86 (283) | 328 (1,078) | 194 (637) |
| B | 181 (595) | 277 (910) | 227 (745) |
| C | 195 (640) | 306 (1,006) | 258 (848) |
| D | 188 (618) | 405 (1,331) | 256 (843) |
| E | 231 (760) | 724 (2,378) | 400 (1,315) |
| F | 231 (760) | 379 (1,246) | 311 (1,023) |
| H | 299 (983) | 552 (1,814) | 362 (1,188) |
| *All velocities are shown as m/s (fps). | | | |

Shear Wave Velocity Profile Selection. Using the P-S velocity logging and seismic CPT results, the applicant developed a Vs profile from the surface to a depth of approximately 182 m (600 ft). The Vs profile in SER Figure 2.5S.4-1 (FSAR Figure 2.5S.4-45, "Shear Wave Velocity Profile - Strata A to J"), identifies the locations of several of the structure foundations as well as the shear wave velocities of the soils. For deeper soil strata, the applicant noted that the shear wave velocity increases from about 304 m/s (1,000 fps) to about 457 m/s (1,500 fps).

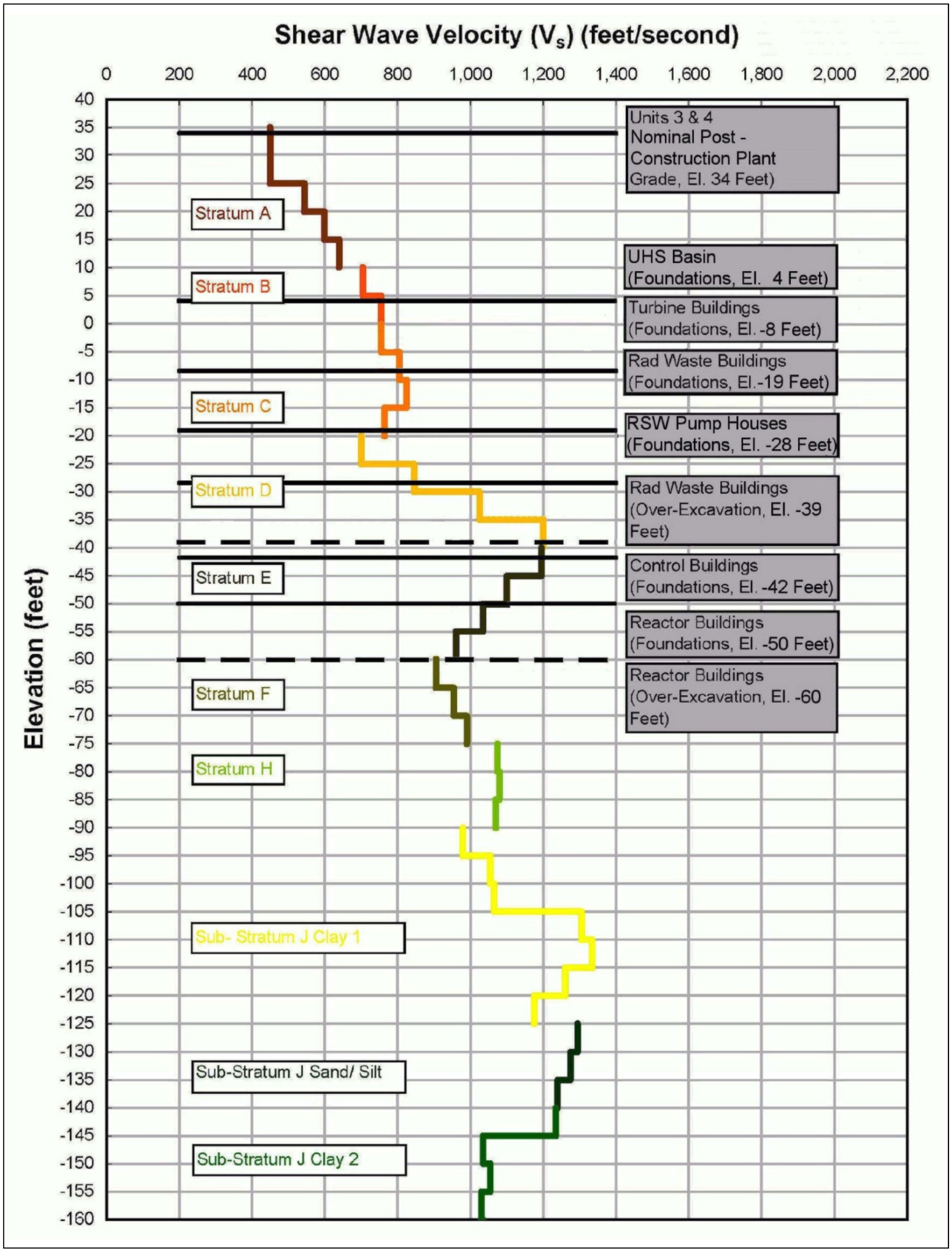


**Figure 2.5S.4-1 Shear Wave Velocity Profile of Strata A to J (FSAR Figure 2.5S.4-45)**

2.5S.4.2.5 Excavation and Backfill

FSAR Subsection 2.5S.4.5, describes the excavation limits, methods of excavation, and monitoring plans to maintain the stability of the excavation. The applicant also describes the construction dewatering requirements and the proposed backfill that will be placed against the below grade nuclear island walls to bring the site to plant grade. The applicant proposed using a combination of excavation slopes and temporary retaining structures to reach the foundation level. Finally, the applicant stated that the backfilling of the excavation will proceed as the below ground structures are completed.

**Sources and Quantity of Backfill and Borrow**

FSAR Subsection 2.5S.4.5.3, describes the sources and quantity of backfill and borrow materials needed to establish site grade within the power block area. The applicant estimated that a total of 4.35 million cubic meters (5.7 million cubic yards) of materials will be moved during earth work at STP, Units 3 and 4. From that total, the applicant will excavate 2.67 million cubic meters (3.5 million cubic yards) of material and import 1.68 million cubic meters (2.2 million cubic yards) of material for use as structural fill. The applicant expected the backfill to come from offsite sources because the excavated soils are not suitable for use as structural fill.

**Extent of Excavations, Fills, and Slopes**

FSAR Subsection 2.5S.4.5.2, describes the extent of excavations, fills, and slopes at the STP, Units 3 and 4, site. The applicant will excavate up to 28.6 m (94 ft) of soil—mostly clays, silts and fine sands—to reach the design's final subgrade elevation of the reactor buildings at -18.36 m (-60.25 ft). The reactor buildings for STP, Units 3 and 4, will be oconcrete fill but other major structures will be directly on dense sand strata or on structural fill. SER Figure 2.5S.4-2 (FSAR Figure 2.5S.4-49A, "Section "A" - Unit 3 Rev. D"), shows cross section A of STP, Unit 3.

<u>Excavation Slopes and Benches</u>. FSAR Subsection 2.5S.4.5.2.2, discusses the applicant's plans for temporary construction slopes of 2 horizontal to 1 vertical (2H:1V) with benches 6.1 m (20 ft) wide, approximately every 6.1 m (20 ft) vertically. However, the applicant stated that these dimensions might change in areas where vertical and horizontal spacing is limited. The applicant performed slope stability analyses and obtained a minimum factor of safety of 1.3 for the temporary excavation slopes.

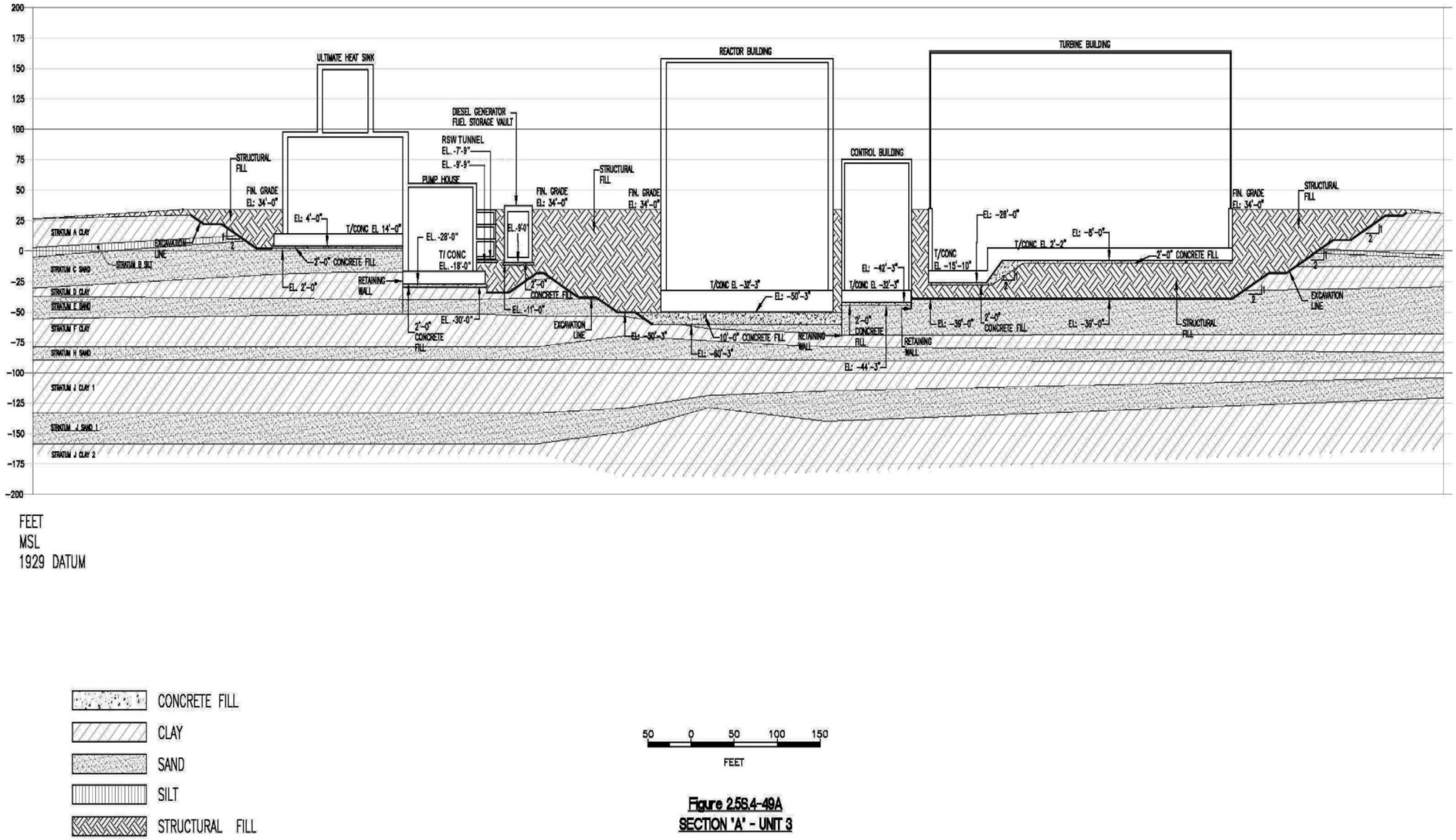


**Figure 2.5S.4-2 Section A Unit 3 (FSAR Figure 2.5S.4-49A)**

Retaining Structures for Adjacent Foundations. The applicant stated that due to abrupt changes in grade in some areas, retaining structures will be used at the STP site.

Reinforced Concrete Retaining Walls. FSAR Subsection 2.5S.4.5.2.4, states that the applicant plans to use reinforced concrete retaining structures at the STP site to facilitate excavation activities. Specifically, the applicant will place these retaining walls on the east side of each reactor building, which will allow crane areas to be at grade and near the building when placing reactor vessels.

Slurry Cut-Off Wall. In FSAR Subsection 2.5S.4.5.2.5, the applicant described the use of a slurry wall to allow the excavation area to be dewatered by hydraulically isolating the excavation inside the wall. The applicant also stated that the slurry wall will be located continuously around the perimeter of the excavations and will have an approximate depth of 38.1 m (125 ft), measured from 1.2 m (4 ft) above the existing water table.

**Compaction Specifications**

In FSAR Subsection 2.5S.4.5.3, the applicant stated that after selecting the structural fill, the material will be tested for index and engineering properties. Following the modified Proctor compaction test procedure, the applicant will compact the structural fill to 95 percent of its maximum dry density and within three percent of its optimum moisture content. The applicant will also prepare the quality control specifications for fill placement and construction monitoring during detailed design.

**Dewatering and Excavation Methods**

FSAR Subsection 2.5S.4.5.4, describes the ground water control system required during construction. The applicant plans to control the ground water by using a dewatering system combined with a perimeter slurry wall. The applicant stated that the dewatering system uses a series of deep wells installed outside of the excavated area and inside the slurry wall combined with sump areas and pumps within the excavated area. Furthermore, the applicant plans to implement measures to prevent runoff down the excavated slopes during heavy rainfall. The applicant will also use sumps, pumps, and other methods to convey water away from the excavation and it will install monitoring wells and piezometers to monitor and evaluate the effectiveness of the dewatering system.

2.5S.4.2.6 Ground Water Conditions

In FSAR Subsection 2.5S.4.6, the applicant described the ground water conditions at the STP, Unit 3 and 4, sites. The applicant provided details of existing ground water conditions and refers to FSAR Section 2.4.12, for additional details.

**Site-Specific Data Collection and Monitoring**

FSAR Subsection 2.5S.4.6.1, states that the ground water is in unconfined conditions in both shallow and deep aquifers. The applicant described the upper water table, which is at an El. of 7.8 m (25.5 ft), as a perched condition that will disappear with the excavation. The applicant selected the ground water level at an El. of 7.8 m (25.5 ft) for liquefaction analysis purposes.

**Construction Stage Dewatering**

The applicant stated that temporary dewatering and the construction of a slurry wall are required during the excavation of the plant foundation and during construction. The applicant plans to lower and maintain the free water and hydrostatic pressures to a minimum of at least 1.5 m (5 ft) below the earth slopes and excavation surfaces. Following the completion of the backfilling stage, the applicant noted that dewatering operations will cease and the ground water will return to normal levels.

**Analysis and Interpretation of Seepage**

The applicant stated that the slurry wall built around the perimeter of the excavation will minimize ground water seepage into the excavation. The applicant also plans to monitor seepage quantities to assess the need for additional dewatering systems.

**Permeability Testing**

The applicant performed slug testing and obtained hydraulic conductivity values of site soils. Although FSAR Table 2.5S.4-23 summarizes these values, the applicant refers to FSAR Section 2.4.12, for a more detailed description.

2.5S.4.2.7 Response of Soil and Rock to Dynamic Loading

FSAR Subsection 2.5S.4.7, addresses the response of soil and rock to dynamic loading. The applicant also addresses COL License Information Item 2.34 in this section and refers to FSAR Section 2.5S.2 for detailed descriptions of the development of the GMRS. The applicant stated that the site-specific soil column extends to the ground surface. Also, the applicant employed the performance-based hazard methodology to develop the GMRS. The applicant referred to FSAR Sections 2.5S.2.5 and 2.5S.2.6, for details of this analysis.

**Shear Wave Velocity (Vs) Profiles**

The applicant measured shear wave and compression wave velocities down to depths of approximately 201 m (660 ft) during the STP, Units 3 and 4, subsurface investigation, although the depth of the subsurface soils is much greater. To supplement the measured velocities, the applicant obtained velocities deeper than 182 m (600 ft) from previous measurements of STP, Units 1 and 2, in addition to oil well logs. The applicant used suspension P-S velocity logging methods and seismic CPT methods to obtain shear and compression wave velocities at STP, Units 3 and 4. SER Figure 2.5S.4-1 (FSAR Figure 2.5S.4-45, “Shear Wave Velocity Profile - Strata A to J”), shows the average shear wave velocity profile for the upper 49 m (160 ft).

The applicant summarized the average shear wave velocities (Vs), Poisson’s ratios (μ), and related parameters in FSAR Table 2.5S.4-27. The applicant made suspension P-S velocity logging measurements at 10 borings, with depths ranging from approximately 61 to 182 m (200 to 600 ft) below the ground surface. The applicant also used the seismic CPT to measure shear wave velocities at five CPTs: three in the STP, Unit 3, area and two in the STP, Unit 4, area, with depths ranging from approximately 19 to 28 m (65 to 95 ft) below the ground surface. Based on the data collected, the applicant concluded that the trends in Vs profiles between the STP, Unit 3, and the STP, Unit 4, areas are generally consistent. The applicant also compared previously obtained shear wave velocity data from the STP, Units 1 and 2, to the STP, Units 3 and 4, data and concluded that the results are relatively consistent within variations of about

±30.48 m/s (±100 fps). The applicant noted one exception between elevations of approximately -12 to -32 m (-40 to -105 ft), but also noted greater differences on the order of 91 to 121 m/s (300 to 400 fps).

Between approximately 182 and 798 m (600 and 2,620 ft) below the ground surface, the applicant obtained shear wave velocity information from the STP, Units 1 and 2, UFSAR. The applicant noted that the subsurface deeper than 182 m (600 ft) below the surface consists of alternating strata of very stiff to hard clay, with some claystones and siltstones and very dense, fine to silty fine sand. The applicant estimated that the top depth of pre-Cretaceous bedrock occurs at approximately 10,515 m (34,500 ft) below the ground surface. The applicant stated that the shear wave velocity profiles below a depth of 182 m (600 ft) increase in shear wave velocity to a depth of approximately 762 m (2,500 ft) below the ground surface and then maintain a similar Vs value of approximately 2,800 m/s (9,200 fps) between depths of 762 and 1,524 m (2,500 and 5,000 ft). The applicant developed three shear wave velocity profile cases that show an increase in shear wave velocity to 2,830 m/s (9,285 fps), at a depth of approximately 762 m (2,500 ft).

To verify the Vs profile for the deeper soils, the applicant searched for geophysical logging results for existing oil wells in the STP site vicinity and selected three wells from the available information. The applicant noted that the deepest sonic logging results extend to a maximum of approximately 4,754 m (15,600 ft) below the ground surface. The applicant converted the sonic logging data to shear wave velocities and notes that the sonic logging data shows generally good agreement with the shear wave profiles in FSAR Figure 2.5S.4-57, "Deep Shear Wave Velocity Profile for the STP Site." In COM 2.5S-1, the applicant committed to provide the refined comparisons of the sonic logging data results and the deep shear wave velocity profiles at a later date.

**Shear Modulus and Damping Curves**

The applicant used dynamic laboratory testing, particularly RCTS tests performed in Strata N clay, N Sand, J Clay, K Clay, M, F, A, and H to obtain data on shear modulus and damping ratio characteristics of site soils over a wide range of strains and to determine the inelastic behavior of the site soils. The applicant also used shear modulus degradation and damping ratio curves from the available literature to characterize the dynamic soil properties.

The applicant developed the generic shear modulus degradation curves for cohesionless soil strata B, C, E, H, J Sand, K Sand/Silt, M, and N Sand using EPRI "Guidelines for Determining Design Basis Ground Motions," (EPRI, 1993) based on strata depths obtained from available literature. SER Figure 2.5S.4-3 (FSAR Figure 2.5S.4-58, "Selected Shear Modulus Degradation Curves for Cohesionless Soil Strata"), depicts the applicant's curves for the cohesionless soil strata and provides a range of values that consider overconsolidation. Similarly, the applicant developed the generic shear modulus degradation curves for cohesive soil strata A, D, F, J Clay, K Clay, L, and N Clay based on the plasticity index (PI) and depth of each strata. The applicant also based the generic damping ratio curves for cohesionless soil strata B, C, E, H, J Sand, K Sand/Silt, M, and N Sand on strata depth. SER Figure 2.5S.4-4 (FSAR Figure 2.5S.4-60, "Selected Damping Ratio Curves for Cohesionless Soil Strata"), depicts the applicant's curves for the cohesionless soil strata. The applicant limited damping to 15 percent.

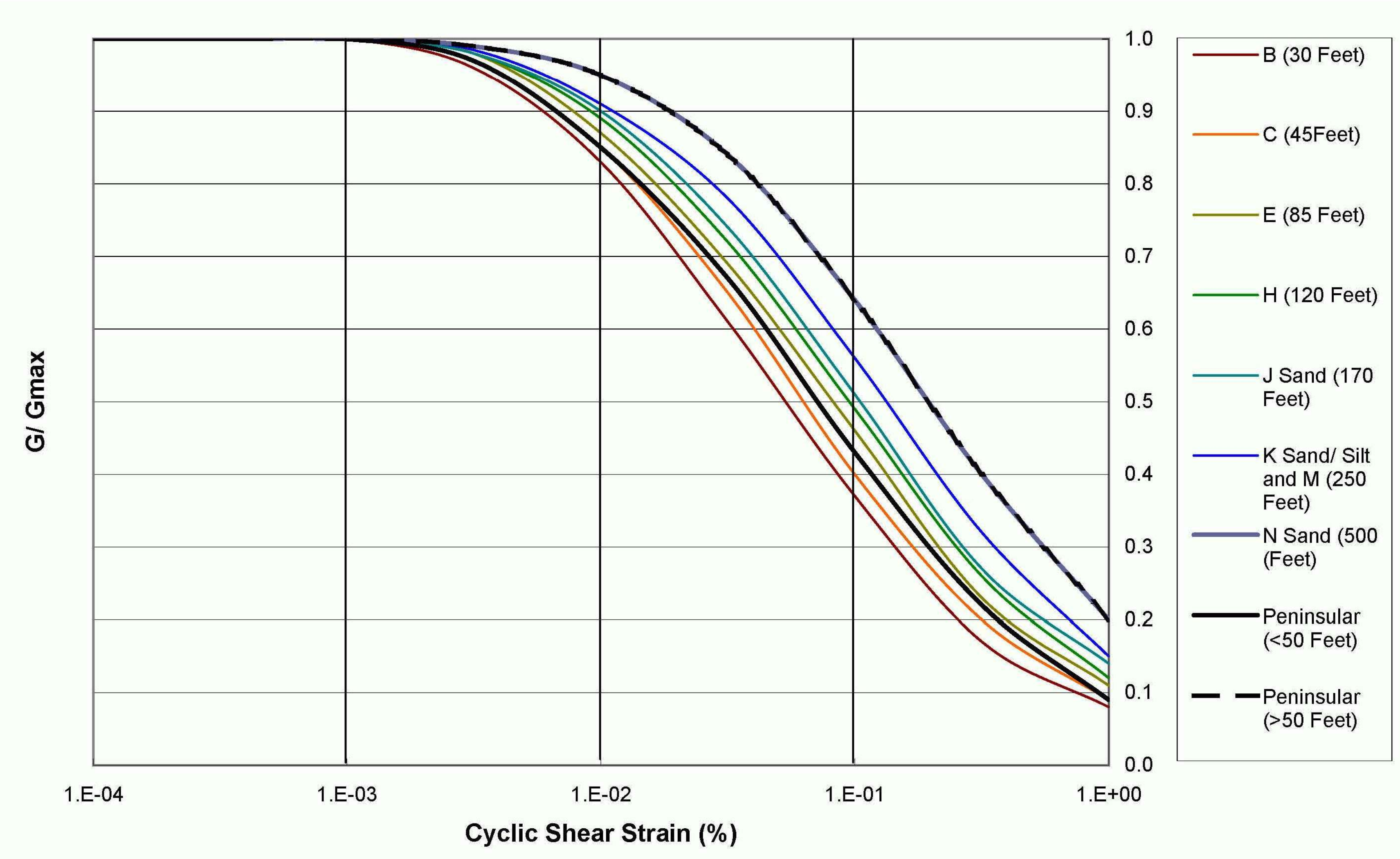


**Figure 2.5S.4-3 Selected Shear Modulus Degradation Curve for Cohesionless Soils (FSAR Figure 2.5S.4-58)**

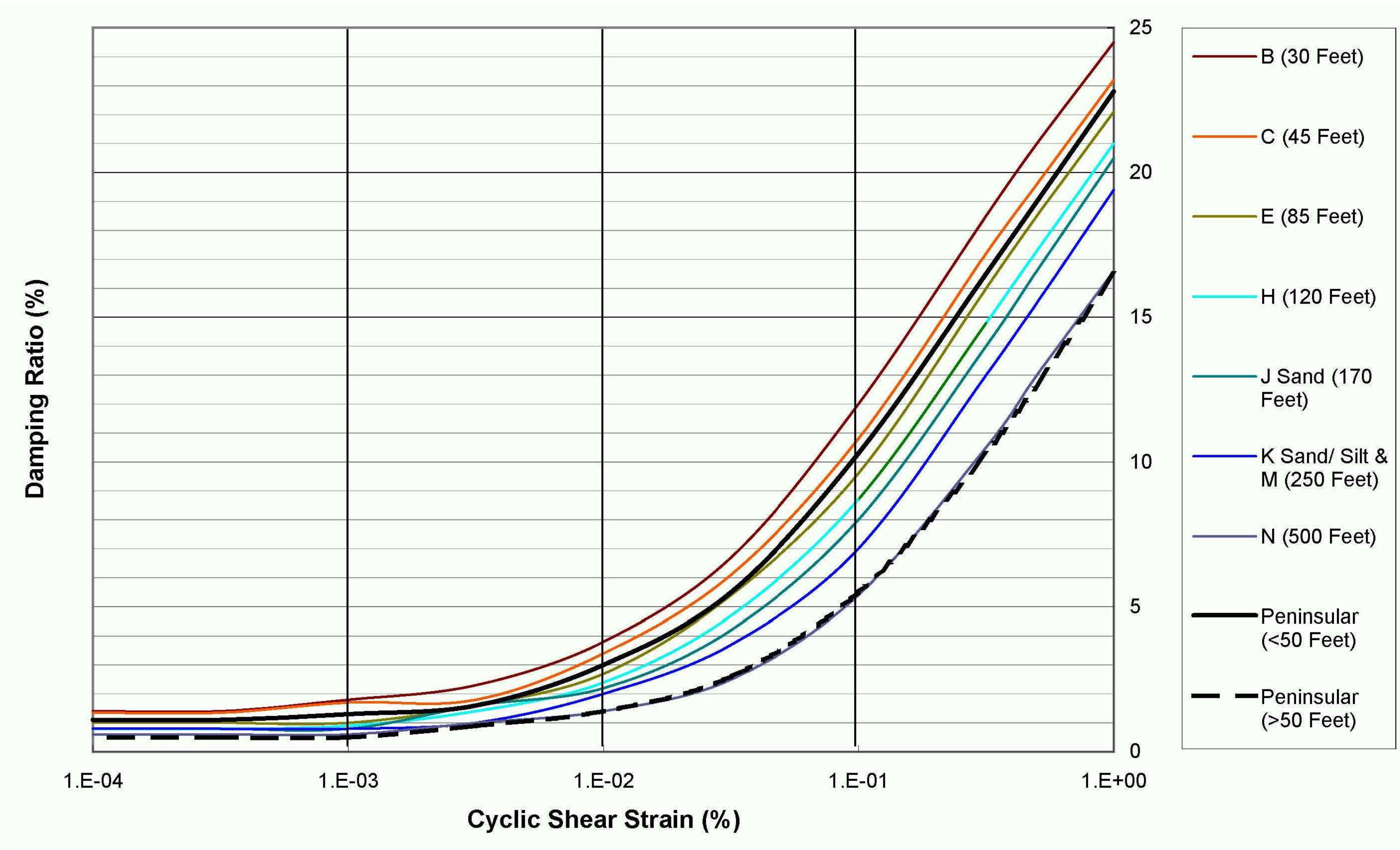


**Figure 2.5S.4-4 Selected Damping Ratio Curve for Cohesionless Soils (FSAR Figure 2.5S.4-60)**

The applicant used a wide range of confining stresses and frequencies in the RCTS tests and, as a result, expected some spread in the results between the site-specific data and the EPRI-based curves (EPRI, 2004). The applicant compared the curves developed from the RCTS results to the EPRI curves and concluded that there was good agreement. However, because the applicant initially performed a limited number of RCTS tests, the applicant commits (COM 2.5S-1) to modify the dynamic soil model if warranted, by further site-specific RCTS test results. To that end, Commitment (COM 2.5S-1) was fulfilled in Revision 3 to the FSAR which includes the results of the applicant's supplemental RCTS tests, as well as additional tests summarized in SER Table 2.5S.4-3.

With regard to the rock, the applicant stated that the top of pre-Cretaceous bedrock occurs at approximately 10,515 m (34,500 ft) below the ground surface. The applicant assumes a damping ratio of 0.2 percent for bedrock and considered the bedrock shear modulus constant in the shear strain range of $10^{-4}$ percent to 1 percent.

Because the applicant has not identified the source of the backfill, RCTS tests were not performed for the backfill materials.

2.5S.4.2.8 Liquefaction Potential

FSAR Subsection 2.5S.4.8, describes the liquefaction potential of the soils at the STP, Units 3 and 4 sites, including the analyses performed and the conclusions reached based on the results.

**Liquefaction Potential of STP, Units 1 and 2**

FSAR Subsection 2.5S.4.8.1, describes the assessment of the liquefaction potential at STP, Units 1 and 2, based on the evaluation of SPT data from the site, including specific borings and cyclic triaxial laboratory test results. The applicant used a peak ground surface acceleration of 0.10 g and an earthquake with a magnitude (M) of 6.0. The applicant concluded that the soils were either not liquefiable or would not liquefy under the assumed seismic conditions.

**Liquefaction Potential of STP, Units 3 and 4**

FSAR Subsection 2.5S.4.8.2, states that the applicant followed Youd et al. (2001) to evaluate the liquefaction potential of the Beaumont formation deposits, which form the upper 182 m (600 ft) of the STP subsurface investigation. The applicant assessed the liquefaction potential primarily for the upper strata at the STP site, including Strata A, B, C, D, E, F, J clay, and K sand/silt. The applicant did not include the backfill materials in the analysis of liquefaction potential. The applicant used the data from three methods—SPT, CPT, and shear wave velocity measurements—to analyze liquefaction potential in the upper 182 m (600 ft). The applicant also stated that the soils deeper than 182 m (600 ft) were geologically old and, therefore, not liquefiable. The applicant noted that the liquefaction analyses based on the three methods did not consider the beneficial effects of age. Finally, the applicant points out that the geologically older deposits tend to have an increased liquefaction resistance, and the high percentage of non-liquefiable soils (typically in the range of 95 percent) supports the conclusion that soil liquefaction at the STP site area is not possible.

<u>Liquefaction Evaluation Methodology</u>. The applicant evaluated liquefaction using empirical methods based on the field data collected from SPT, CPT, and shear wave velocity

measurements. The SPT measurement method is the most developed and the most recognized of the three methods. The applicant calculated the cyclic stress ratio (CSR) (a measure of the stress imparted to the soils by the ground motion) and the cyclic resistance ratio (CRR) (a measure of the resistance of soils to the ground motion). The applicant then used the two ratios to determine the factor of safety. The following paragraphs review the results of the liquefaction potential analysis.

Factor of Safety against Liquefaction. The applicant used the Chinese Method (Youd et al., 2001) to evaluate the liquefaction potential of the clay strata and concluded that the clay strata are not liquefiable. For the remaining soils, the applicant followed the method of ASCE (1980) using SPT, CPT, and Vs data to evaluate the factor of safety, although the method using the SPT data is the most developed and recognized. The applicant analyzed each SPT data point obtained from the borings made inside and outside of the power block area using the liquefaction analysis method proposed by Youd et al. (2001). According to the applicant, a total of 15 tests had a factor of safety less than 1.10. Based on an analysis of the data points with a factor of safety less than 1.10, the applicant concluded that none of the tests will impact the safety on the site. This conclusion reflects the following possibilities: samples were either obtained in areas that will be excavated or in areas where no structures will be emplaced; liquefiable results were surrounded by soils having a high factor of safety against liquefaction; or the tests occurred in a clay stratum that will not liquefy.

The applicant also analyzed each CPT data point obtained from all CPT soundings made inside and outside of the power block area using the liquefaction analysis method proposed by Youd et al. (2001). Following this method, the applicant used uncorrected CPT tip resistance values to obtain a clean sand equivalent, which was then used to calculate the CRR. The applicant noted that the results of the liquefaction analysis based on CPT data show that of 4,489 tests performed at the STP site, 153 resulted in a factor of safety of less than 1.10. Because the samples were obtained from areas that will be excavated, areas where no structures are to be placed, or areas in a clay stratum, the applicant did not expect the materials to liquefy.

The applicant analyzed the Vs data obtained from all of the borings and CPTs made inside and outside of the power block area using the method of Youd et al. (2001). Following this method, the applicant used uncorrected Vs values to calculate a CRR. Based on Vs1 (the shear wave velocity measured in the field and normalized to 1 atmosphere), and the threshold value of Vs1* (the normalized shear wave velocity at and above soils too dense to liquefy), the applicant noted that Vs1* varies from 215 m/s (705 fps) for clean sands to 200 m/s (656 fps) for sands with fine content approaching 35 percent. The applicant stated that approximately 71.6 percent of the 1,687 tests performed demonstrated $V_{s1} \geq V_{s1}^*$, implying that most of the site soils are too dense to liquefy. Based on these results, the applicant concluded that none of the tests will affect safety-related structures at the site, because the samples with a low factor of safety were obtained in areas that will be excavated, areas where no structures will be placed, or areas in clay strata that are not expected to liquefy.

### 2.5S.4.2.9 Earthquake Design Basis

FSAR Subsection 2.5S.4.9, refers to FSAR Subsection 2.5.2.6, where the applicant derives and discusses the horizontal and vertical GMRS.

2.5S.4.2.10 Static Stability

**STP, Units 1 and 2, Foundations**

In FSAR Subsection 2.5S.4.10.1, the applicant described the previous experience with STP, Units 1 and 2. The applicant stated that for the previous units, the factor of safety for bearing capacity was on the order of 3 and the settlement ranged from 5 to 7.6 cm (2 to 3 in.) for both the predicted and the measured settlement, after the recovery of the 8.8 to 12.7 cm (3.5 to 5 in.) of heave.

**STP, Units 3 and 4 Foundations, Subsurface Conditions, and Soil Properties**

In FSAR Subsection 2.5S.4.10.2, the applicant described the subsurface conditions and soil properties used to analyze safety-related seismic Category I foundations. FSAR Table 2.5S.4-16, "Summary of Average Geotechnical Engineering Parameters," summarizes the geotechnical engineering parameters used in this analysis. Structural fill properties are not yet available, so the applicant assumed soil properties from Revision 13 to the STP, Units 1 and 2, UFSAR. The applicant listed the foundation dimensions, founding elevations, and estimated footing pressures for seismic Category I structures. SER Figure 2.5S.4-5 (FSAR Figure 2.5S.4-71, "Adopted Subsurface profiles for the STP 3 & 4 Reactor Buildings"), shows the subsurface profiles of the reactor buildings at STP, Units 3 and 4.

**STP, Units 3 and 4 Bearing Capacity Evaluation**

The ABWR Tier 1 requirement for the minimum bearing capacity supporting the reactor and control buildings is 718 kPa (15 ksf) at the foundation level. The ABWR Tier 1 requirement also states that the remaining safety-related structures should have an adequate bearing capacity. The applicant used Hansen's bearing capacity equations to determine the bearing capacity for the safety-related structures and estimated a pressure for bearing calculations of 718 kPa (15 ksf), the same as the minimum bearing capacity criteria of the ABWR DCD for both the Reactor and Control Buildings. The applicant stated that Hansen's equations are similar to the Terzaghi equation for bearing capacity, except that Hansen's formulation includes foundation shape factors, foundation depth factors, and a reduction factor for large footings. The applicant averaged the shear strength parameters as a simplified way to meet Hansen's method assumption of uniform shear strength in the deformation zone. The applicant achieved a minimum factor of safety of 3 in every case for each safety-related structure.

The applicant stated that the ultimate bearing capacity under seismic loads assumed total stress parameters for the clay strata, effective stress parameters for the sand strata, and the same bearing capacity equations and factors used for the static case. The applicant used a reduced foundation width and length to account for the eccentricity caused by the seismic loading. Based on this calculation, the applicant obtained a factor of safety of 1.5 and found it acceptable for dynamic conditions.

**Settlement**

In FSAR Subsection 2.5S.4.10.4, the applicant described the pseudo-elastic method and the one-dimensional consolidation method of analysis used to estimate settlement. The applicant stated that the pseudo-elastic approach is suitable for granular deposits and clay strata because the clay strata are overconsolidated. The applicant noted that for the most part, the preconsolidation pressures are not exceeded.

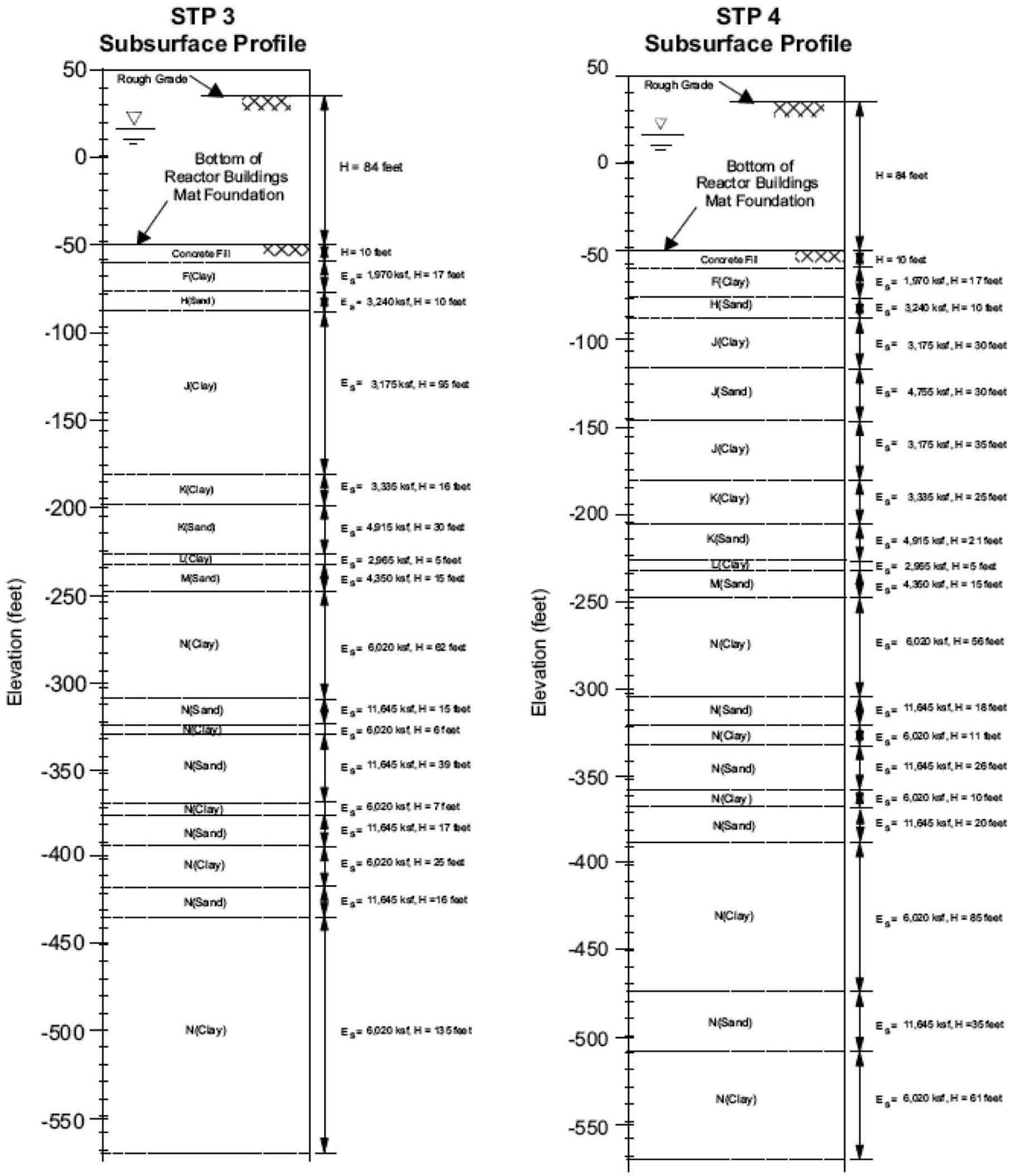


**Figure 2.5S.4-5 Adopted Subsurface Profiles for STP Units 3 and 4 Reactor Buildings (FSAR Figure 2.5S.4-71)**

However, in those instances where the applied stresses exceeded the preconsolidation pressure, the applicant used consolidation theory to compute the settlements. The applicant also stated that the applied pressures only exceeded the preconsolidation pressures in the dewatered state.

The applicant calculated the induced stresses assuming rectangular, flexible foundations and a Boussinesq-type stress distribution. The applicant also used a formulation that allowed for the addition of overlapping stresses from adjacent structures. In addition, the applicant assumed that the concrete fill below the reactor building was incompressible. To ensure that all of the contributions of additional stress from the surrounding buildings were captured in the settlement analysis, the applicant carried the settlement analysis down to a depth of 762 m (2,500 feet). FSAR Table 2.5S.4-42. "Estimated Foundation Settlements." summarizes the estimated settlement results calculated at the center, corner, and middle edges of the foundation mats. The applicant stated that these are maximum settlements, and the buoyancy effects after rewatering will significantly reduce the calculated settlements. A sample calculation for the reactor building used an assumed water table elevation of 5.1 m (17 ft). The results indicate that settlements will be in the range of 60 percent of the maximum settlements calculated for the dewatered state. The applicant also noted that settlements were based on the assumption of a flexible mat, which produces overly conservative results compared to a rigid mat. The applicant introduced a correction methodology to estimate rigid foundation settlements from flexible foundation settlements based on design guidance found in Bowles (1997). The applicant stated that the rigidity of the superstructure can reduce the differential settlements within the mat to half of the differential settlement calculated for the flexible case.

The applicant noted that the industry-accepted criteria for tilt/angular distortion are on the order of 1/300 for frame buildings and 1/750 for buildings supporting sensitive machinery. The applicant computed the angular distortion and tilt for the safety-related structures at the maximum calculated differential settlements from the center to the middle edge for the flexible foundation case. All of the structures were within the 1/300 acceptable limit criteria. The calculated angular distortions exceeded the 1/750 criteria for the reactor buildings, control buildings, UHS basins, RSW tunnels, and diesel generator fuel oil storage vaults (numbers 2 and 3). However, the applicant noted that even for a flexible foundation, the angular distortion/tilt will be within acceptable limits of greater than the 1/750 criteria, because half of the expected total settlement will occur before the placing the equipment in the structures.

The applicant compared the estimated foundation settlements between those calculated for STP, Units 1 and 2, with those for STP, Units 3 and 4. The applicant stated that the greater settlement estimated for STP, Units 3 and 4, is caused by the higher applied loading and larger foundation mat dimensions. Although the ABWR DCD does not specify a Tier 1 settlement requirement, the applicant estimated that the total settlement for the Reactor and Control Buildings would vary between 25.6 to 27.1 cm (10.1 and 10.7 in.) and 19.8 to 21.0 cm (7.8 to 8.3 in.), respectively. SER Table 2.5S.4-6 presents the estimated foundation settlements for key structures at STP, Units 3 and 4.

**Table 2.5S.4-6 Estimated Foundation Settlements (FSAR Table 2.5S.4-42)**

| Structure | | Max Differential Settlement cm (in.) | Max Angular Distortion |
|---|---|---|---|
| Reactor Buildings | Unit 3 | 4.67 (1.84) | 1/600 |
| | Unit 4 | 3.83 (1.51) | 1/750 |

| Structure | | Max Differential Settlement cm (in.) | Max Angular Distortion |
|---|---|---|---|
| Control Buildings | Unit 3 | 4.47 (1.76) | 1/450 |
| | Unit 4 | 5.00 (1.97) | 1/400 |
| UHS Basins | Unit 3 | 5.46 (2.15) | 1/700 |
| | Unit 4 | 5.74 (2.26) | 1/650 |
| RSW Pump Houses | Unit 3 | 1.21 (0.48) | 1/750 |
| | Unit 4 | 1.24 (0.49) | 1/700 |
| RSW Tunnels | Unit 3 | 12.64 (4.98) | 1/700 |
| | Unit 4 | 12.62 (4.97) | 1/700 |
| Diesel Generator Fuel Oil Storage Vault No. 1 | Unit 3 | -1.19 (-0.47) | 1/1000 |
| | Unit 4 | -1.16 (-0.46) | 1/1050 |
| Diesel Generator Fuel Oil Storage Vault No. 2 | Unit 3 | 1.24 (0.49) | 1/500 |
| | Unit 4 | 1.14 (0.45) | 1/550 |
| Diesel Generator Fuel Oil Storage Vault No. 3 | Unit 3 | 0.96 (0.38) | 1/650 |
| | Unit 4 | 0.96 (0.38) | 1/750 |

**Earth Pressures**

FSAR Subsection 2.5S.4.10.5, describes the estimates for static and seismic lateral earth pressures for the plant's below-ground walls. Because the backfill has not been selected, the applicant provided generic calculations that considered active and at-rest pressures but not passive pressures. Lateral earth pressure calculations were based on Rankine earth pressure coefficients, a surcharge pressure of 23.9 kPa (500 psf), backfill unit weight (γ) of 1,922 kg/m$^3$ (120 pcf), and drained friction angle (Φ') of 30°. The applicant stated that the Mononobe-Okabe (M-O) method does not provide realistic results because of the assumption of wall movement, so the applicant calculated the seismic at-rest earth pressures acting against below-grade structural walls using Ostadan (2004). The Ostadan method is based on the assumption of non-yielding walls, which is a more realistic assumption given the rigidity of the structure. The applicant used the soil Vs and the damping that was used for the seismic site-response analysis to derive the spectral acceleration that was applied to the base of the structure. The applicant also calculated lateral forces from the mass of the soil times the spectral acceleration integrated along the height of the wall.

Sample Earth Pressure Diagram. FSAR Figures 2.5S.4-76, "Sample Active Lateral Earth Pressure Diagrams," and 2.5S.4-77, "Sample At-Rest Lateral Earth Pressure Diagrams," depict the static and dynamic lateral earth pressures for the active and at-rest conditions, respectively, for a wall with a maximum height of 25.9 m (85 ft), with the following assumptions:

- level ground surface,
- ground water level at the ground surface,
- Φ'=30° (Backfill),

- ($\gamma$)=1,922 kg/m$^3$ (120 pcf) (Backfill),
- PGA=0.10 g, and
- Uniform Surcharge=23.9 kPa (500 psf).

Until the actual backfill properties and surcharge loads are known, the applicant presented the active and at-rest pressure diagrams for illustration purposes only. SER Figure 2.5S.4-6 (FSAR Figure 2.5S.4-76) illustrates a sample diagram showing the active lateral earth pressures.

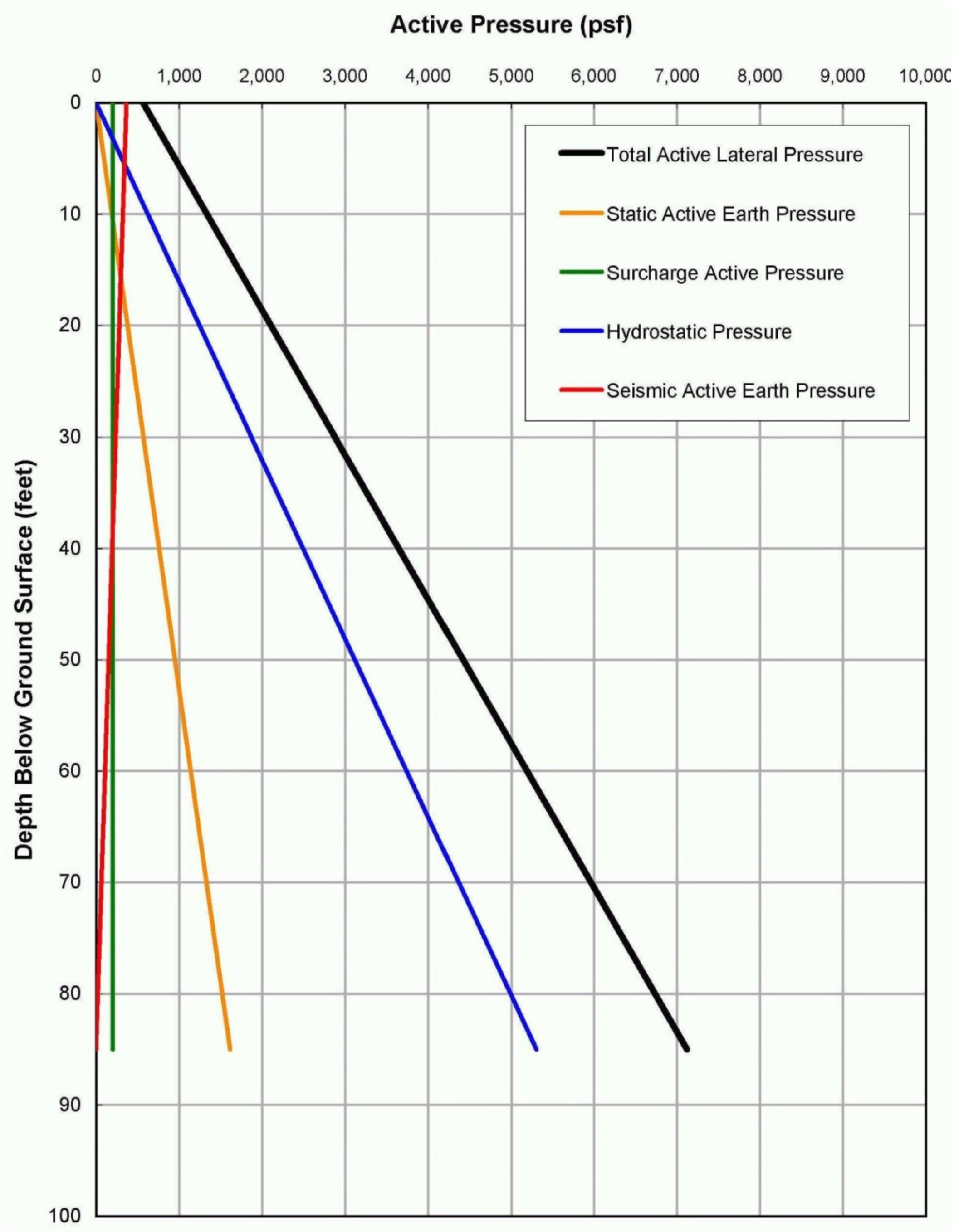

**Figure 2.5S.4-6 Sample Active Lateral Earth Pressure Diagram (FSAR Figure 2.5S.4-76)**

2.5S.4.2.11 Design Criteria

FSAR Subsection 2.5S.4.11, summarizes the geotechnical design criteria discussed in the previous sections of the FSAR. FSAR Subsection 2.5S.4.8, specifies that the acceptable factor of safety against the liquefaction of site soils should be higher than 1.1. FSAR Subsection 2.5S.4.10, presents the bearing capacity and settlement criteria. For the static bearing capacity case and to prevent the uplift of buried pipes, the applicant designed to a minimum factor of safety of 3. For the case of transient earthquake loading, the applicant designed to a factor of safety equal to 2.25 for the dynamic bearing capacity. FSAR Subsection 2.5S.4.10, also specifies a factor of safety of 1 for lateral earth pressures, and a factor of safety of 1.1 for the case of sliding along the base and overturning caused by transient lateral loads.

2.5S.4.2.12 Techniques to Improve Subsurface Conditions

FSAR Subsection 2.5S.4.12, states that due to adequate subsurface conditions at foundation depths, special ground improvements are not necessary. However, the applicant described plans to overexcavate beneath the reactor building, control building, radwaste building, and turbine building and to replace natural soils with structural fill beneath the radwaste, turbine, and control buildings and concrete fill beneath the reactor buildings for improved foundation bearing.

### *2.5S.4.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the stability of subsurface materials and foundations, and the associated acceptance criteria, are in Section 2.5.4 of NUREG–0800. The acceptance criteria for reviewing COL License Information Items 2.26, 2.28, 2.29, 2.30, 2.31, 2.33, 2.34, 2.35, 2.36, 2.37, 2.38, and 2.39 are in Section 2.5.4 of NUREG–0800.

In particular, the applicable regulatory requirements for reviewing geologic and seismic information are as follows:

- 10 CFR 50.55a, Codes and standards requires that structures, systems, and components (SSCs) be designed, fabricated, erected, constructed, tested, and inspected in accordance with the requirement of applicable codes and standards commensurate with the importance of the safety function to be performed.

- 10 CFR Part 50, Appendix A, GDC 1, “Quality standards and records,” requires that SSCs important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. The criterion also requires that appropriate records of the design, fabrication, erection, and testing of SSCs important to safety be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit.

- 10 CFR Part 50, Appendix A, GDC 2, “Design bases for protection against natural phenomena,” as it relates to consideration of 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 Part 50, Appendix A, GDC 44, "Cooling water," requires that a system be provided with the safety function of transferring the combined heat load from SSCs important to safety to a UHS under normal operating and accidental conditions.

- 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants," establishes quality assurance requirements for the design, construction, and operation of those structures, systems, and components of nuclear power plants that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.

- 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," as it applies to the design of nuclear power plant structures, systems, and components important to safety to withstand the effects of earthquakes.

- 10 CFR Part 100, "Reactor Site Criteria," provides the criteria that guide the evaluation of the suitability of proposed sites for nuclear power and testing reactors.

- 10 CFR 100.23, "Geologic and seismic siting criteria," provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and designing of nuclear power plants.

The related acceptance criteria are summarized from SRP Section 2.5.4, "Stability of Subsurface Materials and Foundations":

- Geologic Features: In meeting the requirements of 10 CFR Parts 50 and 100, the section defining geologic features is acceptable if the discussions, maps, and profiles of the site stratigraphy, lithology, structural geology, geologic history, and engineering geology are complete and are supported by site investigations sufficiently detailed to obtain an unambiguous representation of the geology.

- Properties of Subsurface Materials: In meeting the requirements of 10 CFR Parts 50 and 100, the description of properties of underlying materials is considered acceptable if state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soils and rocks in the site area.

- Foundation Interfaces: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of the relationship of foundations and underlying materials is acceptable if it includes: (a) a plot plan or plans showing the locations of all site explorations, such as borings, trenches, seismic lines, piezometers, geologic profiles, and excavations with the locations of the safety-related facilities superimposed thereon; (b) profiles illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials; (c) logs of core borings and test pits; and (d) logs and maps of exploratory trenches in the application for a COL.

- Geophysical Surveys: In meeting the requirements of 10 CFR 100.23, the presentation of the dynamic characteristics of soil or rock is acceptable if geophysical investigations were performed at the site and the results obtained are presented in detail.

- Excavation and Backfill: In meeting the requirements of 10 CFR Part 50, the presentation of the data concerning excavation, backfill, and earthwork analyses is acceptable if: (a) the sources and quantities of backfill and borrow are identified and evidence shows adequate investigations by borings, pits, and laboratory property and strength testing (dynamic and static) and these data are included, interpreted, and summarized; (b) the extent (horizontally and vertically) of all seismic Category I excavations, fills, and slopes are clearly shown on plot plans and profiles; (c) compaction specifications and embankment and foundation designs are justified by field and laboratory tests and analyses to ensure stability and reliable performance; (d) the impact of compaction methods are incorporated into the structural design of the plant facilities; (e) quality control methods are discussed and the quality assurance program described and referenced; (f) the control of ground water during excavation to preclude degradation of foundation materials and properties is described and referenced.

- Ground Water Conditions: In meeting the requirements of 10 CFR Parts 50 and 100, the analysis of ground water conditions is acceptable if the following are included in this subsection or cross-referenced to the appropriate subsections in SRP Section 2.4 of the SAR: (a) discussion of critical cases of ground water conditions relative to the foundation settlement and stability of the safety-related facilities of the nuclear power plant; (b) plans for dewatering during construction and the impact of the dewatering on temporary and permanent structures; (c) analysis and interpretation of seepage and potential piping conditions during construction; (d) records of field and laboratory permeability tests as well as dewatering induced settlements; (e) history of ground water fluctuations as determined by periodic monitoring of 16 local wells and piezometers.

- Response of Soil and Rock to Dynamic Loading: In meeting the requirements of 10 CFR Parts 50 and 100, descriptions of the response of soil and rock to dynamic loading are acceptable if: (a) an investigation was conducted and discussed to determine the effects of prior earthquakes on the soils and rocks in the vicinity of the site; (b) field seismic surveys (surface refraction and reflection and in-hole and cross-hole seismic explorations) are accomplished and the data presented and interpreted to develop bounding P and S wave velocity profiles; (c) dynamic tests were performed in the laboratory on undisturbed samples of the foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysterietic damping properties of the soils and the results are included.

- Liquefaction Potential: In meeting the requirements of 10 CFR Parts 50 and 100, if the foundation materials at the site adjacent to and under seismic Category I structures and facilities are saturated soils and the water table is above bedrock, then an analysis of the liquefaction potential at the site is required.

- Static Stability: In meeting the requirements of 10 CFR Parts 50 and 100, the discussions of static analyses are acceptable if the stability of all safety-related facilities was analyzed from a static stability standpoint, including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, and lateral loading conditions.

- Design Criteria: In meeting the requirements of 10 CFR Part 50, the discussion of criteria and design methods is acceptable if the criteria used for the design, the design methods employed, and the factors of safety obtained in the design analyses are described and a list of references are presented.

- Techniques to Improve Subsurface Conditions: In meeting the requirements of 10 CFR Part 50, the discussion of techniques to improve subsurface conditions is acceptable if plans, summaries of specifications, and methods of quality control are described for all techniques to be used to improve foundation conditions (such as grouting, vibroflotation, dental work, rock bolting, or anchors).

In addition, the geologic characteristics should be consistent with the appropriate sections from RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants”; RG 1.28, “Quality Assurance Program Requirements (Design and Construction)”; RG 1.132, RG 1.138; RG 1.198, “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites”; and RG 1.206.

#### *2.5S.4.4 Technical Evaluation*

The staff reviewed the information in Section 2.5S.4 of the STP, Units 3 and 4, COL FSAR:

*COL License Information Items*

- COL License Information Item 2.26 Stability of Subsurface Material and Foundation
- COL License Information Item 2.28 Field Investigations
- COL License Information Item 2.29 Laboratory Investigations
- COL License Information Item 2.30 Subsurface Conditions
- COL License Information Item 2.31 Excavation and Backfilling for Foundation Construction
- COL License Information Item 2.32 Ground Water Conditions
- COL License Information Item 2.33 Liquefaction Potential
- COL License Information Item 2.34 Response of Soil and Rock to Dynamic Loading
- COL License Information Item 2.35 Minimum Static Bearing Capacity
- COL License Information Item 2.36 Earth Pressures
- COL License Information Item 2.37 Soil Properties for Seismic Analysis of Buried Pipes
- COL License Information Item 2.38 Static and Dynamic Stability of Facilities
- COL License Information Item 2.39 Subsurface Instrumentation

The staff evaluated the above list of COL license information items in the following subsections.

2.5S.4.4.1 Description of Site Geologic Features

FSAR Subsection 2.5S.4.1 refers to FSAR Subsections 2.5S.1.1 and 2.5S.1.2, for detailed descriptions of the regional geology and site geology, respectively. SER Subsections 2.5S.1.4.1 and 2.5S.1.4.2, provide the technical evaluation of the regional and site geologic features.

2.5S.4.4.2 Properties of Subsurface Materials

SER Subsection 2.5S.4.2.2, summarizes FSAR Subsection 2.5S.4.2. The applicant performed an exploratory program that included SPTs, CPTs, undisturbed sampling, test pits, field testing, geophysical surveys, and laboratory testing. The applicant stated that the recommendations of RG 1.132 and RG 1.138, guided the exploratory and laboratory testing programs, respectively. The soil properties are used as input to the engineering analyses performed to establish the safety of the structure foundations. The staff reviewed FSAR Subsection 2.5.4.2, and noted several areas where there was a need for additional information or clarification to ensure complete and accurate soil property characterizations.

## Description of Subsurface Materials

FSAR Table 2.5S.4-16, reveals that the applicant did not obtain soil properties for Stratum M, including SPT N-values. The applicant measured shear wave velocities in Stratum M at less than the shear wave velocity of Stratum K. The staff issued RAI 02.05.04-6, requesting the applicant to justify the assumed N-values and soil properties based on Stratum K given the differences in shear wave velocity between the two strata.

In its response to RAI 02.05.04-6, dated July 9, 2008 (ML081960070), the applicant demonstrated that it derived the SPT N-value for Stratum M using the relationship for high strain elastic modulus, E=36 N ksf, which correlates the high strain elastic modulus derived from the small strain shear wave velocity measurements with the SPT N-value. From this relationship, the applicant back-calculated an N of 36, which is slightly larger than the N-value of 30 assumed for Stratum M. Having determined the N-value, the applicant used a relationship between the N-value and the friction angle to determine a Φ value of 33°. Because empirical relationships indicate a friction angle of 35 to 40° for N-values greater than 30, the applicant concluded that this Φ value is conservative. The applicant borrowed the remaining material properties from Stratum K Sand including a moisture content of 21, a fines content of 45 percent, and a unit weight of 2,034 $kg/m^3$ (127 pcf), which are also similar to Stratum J Sand. Although the shear wave velocities measured in the three sand strata of 388 m/s (1,275 fps) for J; 417 m/s (1,370 fps) for K; and 355 m/s (1,165 fps) for M are slightly different, the applicant determined that it is reasonable to assume that all three strata have similar properties.

The staff reviewed the boring logs for B-305 DH and B-405 DH, which recorded the interval corresponding to Stratum M as sand (USCS SP-SM) in both borings and Stratum K as sand (SP-SM) in B-305 DH and silt (ML) in B-405 DH. Accordingly, the staff agreed with the applicant's conclusion that Stratum M corresponds to Stratum K Sand in boring B-305 DH. The applicant noted that the high N-value of greater than 30 indicates that the stratum is very dense. The staff agreed with the applicant that the assumed unit weights and moisture content are reasonable and in agreement with measured values in Strata K and J sand. Given the variability that occurs with Stratum M across the site, the applicant assumed a higher fines content for Stratum M, which the staff found acceptable. Also, given the dense nature of the sand, the staff concluded that Stratum M will not undergo very much compression, and drainage is therefore not an issue. The staff also agreed with the applicant that liquefaction is unlikely in dense sand with a low seismic demand. The staff also concluded that the back-calculated friction angle of 33° is conservative for a dense sand. Finally, the staff concluded that the approach for determining material properties of Stratum M is adequate. Therefore, RAI 02.05.04-6 is resolved and closed.

The staff noted that the FSAR contains little information describing the presence of fissures or slickensides in the Beaumont clay, even though Mahar and O'Neil (1983) describe the difficulties in measuring the soil properties of fissured clays in the Beaumont. The staff issued RAI 02.05.04-22, requesting the applicant to provide a thorough discussion of the desiccation features encountered in the Beaumont Clay including: (1) how the desiccation features compare to those discussed in Mahar and O'Neil (1983); and (2) how the laboratory and in situ test results are conservative in the evaluation of engineering properties used for bearing capacity, slope stability, and settlement analyses.

In its response to RAI 02.05.04-22, dated July 20, 2009 (ML092030132), the applicant described the encountered fissures, slickensides, and calcareous deposits in the samples taken at the site, are consistent with the soils documented in Mahar and O'Neil (1983). The applicant explained that these features can cause samples to fail prematurely along planes of weakness, which indicate lower strength than what occurs in situ. The applicant also noted that stress release from sampling leads to sample disturbance, which can have a detrimental effect on the accuracy of measured soil properties compared to properties in situ. The applicant also indicated that although soil samples may not provide accurate results, the strength and compressibility of the soil determined from laboratory tests would be conservative. The applicant reasoned that the collective use of in situ tests, SPTs, CPTs, and seismic measurements together with the measured laboratory strength and compressibility test results provide an accurate assessment of the mass properties of the soil. The applicant used the properties derived from a consideration of all the test data, laboratory and in situ, in the stability analyses.

The staff reviewed the applicant's overall approach for choosing soil properties to use in the engineering analyses, as well as the recommendations of Mahar and O'Neil (1983). The staff concluded that the applicant has conservatively selected strength parameters for the engineering analyses using the appropriate means. Additionally, the staff concluded that as the structure is incrementally placed, the higher stresses should further compress the Beaumont clay resulting in strength gains that will improve site stability. Therefore, RAI 02.05.04-22 is resolved and closed.

**Overconsolidation Ratios**

The applicant used field and laboratory tests to determine the soil compressibility for settlement analyses. One factor affecting compressibility is the overconsolidation ratio (OCR). The OCR is the ratio of the maximum past pressure to the present effective overburden pressure. A soil with an OCR greater than 1.0 will experience less settlement than a normally consolidated soil with an OCR equal to 1.0 when subjected to the same foundation load. The applicant employs two methods to determine the OCR: an interpretation of CPT data and an interpretation of consolidation test data. The staff asked two clarifying questions regarding OCR values to ensure that the soil properties used in settlement analyses were appropriate.

FSAR Figures 2.5S.4-29, "Overconsolidation Ratio (OCR) From CPT Data (STP 3)," and 2.5S.4-30, "Overconsolidation Ratio (OCR) From CPT Data (STP 4)," illustrate how the calculated OCRs vary within the clay strata but generally decrease with depth. The applicant selected an average OCR of 1.8 in Stratum F at STP, Unit 3, and in J Clay 1 at STP, Unit 4, although some data points have an OCR of less than 1. The staff noted that the consolidation test data do not show this trend at similar elevations in FSAR Figure 2.5.4-28, "Laboratory Test Results -Preconsolidation Pressure (Pc') versus Elevation." The staff issued RAI 02.05.04-7, asking the applicant: (1) to explain the difference in the results between the OCR data determined from field data and the consolidation test data, and (2) to justify the assumed OCR of 1.8.

In its response to RAI 02.5.04-7, dated July 9, 2008 (ML081960070), the applicant noted that in most soil profiles, the OCR decreases with an increase in depth, eventually reaching unity or very close to unity. The applicant computed the OCR values using a third order equation and the estimated shear strength derived from the cone tip resistance. The applicant expected a reasonable amount of scatter using this empirical equation resulting in occasional OCR values of less than 1. The applicant concluded that the average values are representative of the OCR values derived from the CPT results. The applicant added that there is no geologic mechanism for an OCR of less than one in Pleistocene-age samples. The applicant also addressed the difference between the consolidation test OCRs and the CPT-derived OCRs by stating that the average OCR values from the CPT for Stratum F are 1.8 for STP, Unit 3 and 2.5 for STP, Unit 4, which gives a rounded-up average of 2.2. Although the average OCR from consolidation tests in Stratum F is 2.9, the applicant selected an OCR value for Stratum F of 2.6 to use for the engineering analysis. Likewise, the applicant stated that the average OCR value from the CPT for Stratum J Clay is 1.8. The average OCR from consolidation tests for Stratum J Clay is 1.9, and the OCR value selected for engineering purposes for Stratum J Clay is 1.7.

The staff reviewed the consolidation test data and confirmed that the applicant's assumed OCR of 2.2 for Stratum F and 1.7 for Stratum J is reasonable. The staff also noted that the use of the third order equation and shear strength derived from the measured tip resistance accounts for the CPT-derived OCR of less than 1. Because the applicant relied on the results of the consolidation tests to select OCR values for the engineering analyses, the staff concluded that the OCRs used in the analyses are conservative and acceptable. Therefore, RAI 02.05.04-7 is resolved and closed.

FSAR Figure 2.5S.4-28, "Laboratory Test Results -Preconsolidation Pressure (Pc') versus Elevation," shows the computed OCR for consolidation tests, which falls below 1.0 at elevations lower than -82 m (-270 ft), even though FSAR Figure 2.5S.4-20, "Atterberg Limits versus Elevation," does not indicate the presence of normally consolidated or underconsolidated strata below this depth. The staff issued RAI 02.05.04-8, asking the applicant to reconcile the differences in these data and explain how the consolidation data were used to compute settlements.

In its response to RAI 02.05.04-8, dated July 9, 2008 (ML081960070), the applicant stated that in most soil profiles, the OCR decreases with increasing depth and eventually reaches unity, or very close to unity. The applicant also explained that at an El. of -82 m (-270 ft), the effective vertical overburden pressure is close to 957 kPa (20 ksf), and the soil is highly consolidated with a typically low natural moisture content. The applicant stated that a very low liquidity index could still indicate soils that are normally consolidated or close to normal consolidation. Because there is no geologic mechanism for Pleistocene-age samples to have an effective overburden pressure greater than the maximum past pressure, the applicant noted that the preconsolidation pressures should not plot below the effective overburden pressure line. The applicant explains that the two deep points lying below the effective overburden pressure line shown on FSAR Figure 2.5S.4-28, are likely due to disturbances from pressure relief during the sampling process. The applicant used elastic parameters to estimate settlement in all of the settlement calculations, except when considering Stratum L at around the El. of -70 m (-230 ft). Finally, the applicant determined that the computed virgin compression settlement beneath any structure is less than 0.635 cm (0.25 in.).

The staff concurred that samples retrieved from a greater depth are subject to significant stress relief that alters the sample before testing, which affects the results of the laboratory consolidation tests. The staff also noted that the effect of stress relief may be greater in soils that are overconsolidated through desiccation, such as the clays of the Beaumont Formation due to defects like fissures. Furthermore, the staff understands that sample disturbance makes it difficult to select the actual preconsolidation pressure with a high degree of accuracy, which may have resulted in the selection of a lower than actual preconsolidation pressure for the two test points in question. Therefore, the staff agreed with the applicant that there is no geologic mechanism for Pleistocene age samples to have a present effective overburden pressure greater than the maximum past pressure, so the OCR should be greater than one at all depths.

The staff also agreed with the applicant that an OCR of 1.0, which represents virgin compression, is conservative to compute the settlement contribution from strata at depth. Accordingly, the staff concluded that the applicant has adequately addressed the anomalous data in FSAR Figure 2.5S.4-28 and RAI 02.05.04-8, is resolved and closed.

**Shear Strength of Clays**

Shear strength parameters of soils are required for bearing capacity determinations as well as lateral stresses on buried walls. Two cases that require analysis are end of construction and long-term conditions. The end of construction case requires the determination of the "undrained" shear strength of the clay soils, which the applicant determined from SPT N-values, CPT tip resistance, and laboratory testing. The staff asked two questions related to the determination of the undrained strength of the clay soils.

FSAR Subsection 2.5S.4.2.1.6, provides the soil undrained shear strength for Stratum F Clay as determined from SPT tests. The applicant corrected the SPT N-values to account for the overburden pressure. However, the staff considered the use of an overburden pressure correction factor unnecessary for clay soils and perhaps unconservative in some instances. The staff was concerned that the bearing capacity of Strata A through D may have been over-estimated as a result of applying the overburden correction to the field SPT N-value. The staff issued RAI 02.05.04-9, asking the applicant to justify correcting the N-values for clay CH and CL soils for the effects of overburden, considering that for overburden pressures of less than 107 kPa (1 ton per square foot [tsf]), the corrected N-value would be unconservative.

In its response to RAI 02.05.04-9, dated August 12, 2008 (ML082270381), the applicant stated that correction factors for overburden pressure and energy were applied to the N-values derived in both the granular and cohesive strata. The applicant used a correction factor developed for granular soils to obtain the $N_{1\ 60}$, which is the SPT N-value normalized to 1 kPa (15 psi) and 60 percent of the theoretical energy of a 63.5-kg (140-lb) weight falling 76.2 cm (30 in.) to strike the anvil that drives the drill rods into the ground to measure the resistance to penetration, N or N-value. The applicant considered the overburden correction for N-values of cohesive soils to be a conservative approach, because it reduced the measured N-value for all soils located below about the mid-point of Stratum C. The applicant also stated that the correction factor for Stratum D ranges from 0.85 to 0.68 from top to bottom. The applicant demonstrated that the correction factor is more conservative when applied to deeper strata. The applicant further notes that for all strata below Stratum A, the undrained strength value derived from the $N_{1\ 60}$ value was significantly less than the undrained shear strength ($S_u$) value selected for design, where the laboratory strength test results and the CPT-derived strength results were the primary data that were used for assigning design shear strength values for the clay strata. The applicant concluded that undrained shear strength derived from the N-values, except for Stratum A, has little impact on the $S_u$ values selected for design. In considering Stratum A, the applicant noted that the $S_u$ value selected for design is less than the $S_u$ value based on the N-value. The applicant concluded that the overburden correction factors result in reduced and therefore conservative N-values for all strata except Stratum A.

The staff concurred that the N-values would be reduced at an overburden pressure below 107 kPa (1 tsf), and the resulting undrained shear strength derived from the reduced N-values would be conservative for all strata or portions of strata where the overburden pressure was greater than 107 kPa (1 tsf). The staff concluded that laboratory shear strength testing and CPT soundings are preferred methods for deriving undrained shear strength in cohesive soils. The staff also noted that the CPT and the laboratory tests derived undrained shear strengths greater than the corrected SPT-derived shear strengths in Stratum C and above, which removes the concern that unconservative shear strengths were used in the design. Therefore, the staff concluded that the use of the overburden correction factor had a conservative effect on the selection of shear strength for the clay strata. Therefore, RAI 02.05.04-9 is resolved and closed.

FSAR Subsection 2.5S.4.2.1.6, also discusses the Stratum F undrained shear strength of 162 kPa (3.4 ksf) based on the results of CPT testing. The applicant appeared to rely more heavily on the CPT-derived shear strength than the shear strengths, and on the estimated shear strengths from correlations with energy-corrected SPT N-values and/or measured laboratory test results from unconfined and UU laboratory triaxial testing. The staff issued RAI 02.05.04-10, requesting the applicant to clarify how CPT shear strength correlations are more credible than laboratory test measurements, since the site-specific cone factor was derived from the laboratory test results.

In its response to RAI 02.05.04-10, dated August 12, 2008 (ML082270381), the applicant stated that the selection of an undrained shear strength value for Stratum F is based on the results of SPTs, CPTs, and laboratory strength tests for STP, Units 3 and 4, in addition to laboratory strength results from STP, Units 1 and 2. The applicant also changed the procedure and corrected the SPT N-value for hammer energy only to obtain the $N_{60}$ value neglecting the correction for the overburden pressure. The applicant used an $N_{60}$ value of 34 to estimate an undrained shear strength of 191 kPa (4.0 ksf). The applicant also used CPT tip resistance and an assumed $N_{kt}$ of 19 to estimate the undrained shear strength, thus calculating an undrained shear strength of 162 to 191 kPa (3.4 and 4.0 ksf) for STP, Units 3 and 4, respectively. Based on the UU triaxial tests, the applicant concluded that the median value of 158 kPa (3.3 ksf) is more realistic than the mean value of 129 kPa (2.7 ksf), because the greater value reduces the influence of three low results that were likely the result of sample disturbance. Based on the SPT, CPT, and UU triaxial tests, the applicant concluded that the 162 kPa (3.4 ksf) design value is reasonable.

The staff agreed that a correction for hammer efficiency is applicable and a correction for overburden pressure on a cohesive soil is not applicable. Therefore, the staff concluded that the undrained shear strength based on the SPT was determined correctly. The staff also noted that the revised calculation changed the undrained shear strength based on SPT N-values from 129 kPa (2.7 ksf) to 191 kPa (4.0 ksf). The staff also concluded that the $N_{kt}$ factor of 19 the applicant used to calculate the shear strength from the CPT cone tip resistance is acceptable because it is based on correlations with site-specific laboratory tests. This leads the staff to further conclude that the CPT-derived undrained shear strengths of 162 to 191 kPa (3.4 and 4.0 ksf) for STP, Units 3 and 4, respectively, are reasonable. The staff further concurred with the applicant that the median value of the 10 UU triaxial tests are more likely than the average value to be representative of the undrained shear strength of Stratum F. Therefore, the staff concludes that the design strengths are best assumed for Stratum F based on the combination of the STP, Units 3 and 4, field and laboratory data, which all provide undrained strength parameters in the range of shear strength selected by the applicant for the design value of 162 kPa (3.4 ksf). Accordingly, RAI 02.05.04-10 is resolved and closed.

In FSAR Subsection 2.5S.4.2.1.6, the applicant determined the drained strength parameters for Stratum F, but then assumed an effective angle of internal friction (Φ) of 20° determined from testing Stratum D soils. The staff issued RAI 02.05.04-11, asking the applicant to clarify why Stratum D test data were used in lieu of Stratum F test data.

In its response to RAI 02.05.04-11, dated August 12, 2008 (ML082270381), the applicant stated that there was an error in reporting the cohesion of Stratum F, Stratum D, and Stratum J Clay. This error was corrected in Revision 3 of the COL application. Additionally, the applicant explained that although the drained strength, as determined from the laboratory tests for Stratum F, is reasonable for that stratum, a friction angle of 20° was used to determine lateral earth pressure coefficients and to compute the lateral stresses on below ground walls. The applicant obtained the friction angle from a table of friction angles for soils interfacing with concrete and/or steel in the Naval Facilities Engineering Command Manual DM 7.02 (NAFVAC, 1986).

The staff reviewed the referenced table in the Naval Facilities Engineering Command Manual DM 7.02 and noted that the range of friction angle values recommended for mass concrete against very stiff to hard preconsolidated clay ranged from 22 to 26˚. The applicant selected a value of 20˚, which was conservative, and more representative of the long-term case than using the cohesion value of 95 kPa (2 ksf) and the friction angle of 8˚, as determined from laboratory drained tests on Stratum F soils. The staff concluded that assuming a friction angle of 20˚ is conservative for computing the lateral earth pressures and acceptable. Therefore, RAI 02.05.04-11 is resolved and closed.

**Soil Compressibility and Elastic Modulus**

In order to perform settlement analyses, the applicant needs to determine the compressibility of the soil, which requires an elastic modulus for soils that will not be stressed beyond the preconsolidation pressure. The applicant calculated the large strain elastic modulus for the site clay strata using an empirical relationship based on Beaumont Clays and a relationship based on small-strain shear wave velocity. The applicant averaged the results using a weighted formula that favored the shear wave velocity-derived value in the ratio of 2:1. The staff issued RAI 02.05.04-17, requesting the applicant to explain why the two methods used to determine the elastic modulus provide different results, and why the shear wave velocity-derived results are favored by 2:1 in computing an average value.

In its response to RAI 02.05.04-17, dated April 1, 2009 (ML090930717), the applicant proposed changes to the FSAR in conjunction with the applicant's response to RAI 02.05.04-13, Supplement 1 dated January 28, 2009, and February 23, 2009 (ML091820695). The applicant computed the empirically-based modulus values using Equations 2.5S.4-4A and 2.5S.4-4B. The applicant then presented the empirically-based modulus values and the velocity-based modulus values corrected for large strain in FSAR Table 2.5S.4-14, "Summary of High Strain Elastic Moduli Estimates." FSAR Table 2.5S.4-14 shows that empirically-based modulus values are compatible with the velocity-based values. The applicant also determined that the small strain modulus based on FSAR Equations 2.5S.4-5 and 2.5S.4-6, from the measurement of shear wave velocities in situ represent the highest achievable stiffness, because they are measured at non-destructive strains. Also, because the small strain elastic modulus represents the maximum stiffness of the stratum, the applicant assigned a weighting of 2:1 in favor of the velocity-derived elastic modulus, as compared to the empirically-derived modulus estimated from undrained shear strength ($S_u$).

The staff reviewed the applicant's response to RAI 02.05.04-17, and concluded that the differences in some strata and the good correspondence in other strata are expected due to the natural variability in the subsurface and the reliance on empirical relationships developed from other sources. The staff considered the estimates of elastic modulus based on the measured shear wave to be the most reliable, because they are not affected by sample disturbance. The staff also noted that averaging the results from the two methods makes the elastic modulus assumed for design more conservative than assuming the shear wave velocity-derived value alone. Therefore, the staff concluded that the applicant's decision to favor the shear wave velocity-derived elastic modulus in the ratio of 2:1, when averaging the results with the empirically based elastic modulus, is reasonable and conservative because it ensured that greater weight was placed on the more reliable, least disturbed in situ measurement. The staff further concluded that the applicant took a conservative approach using appropriate field data and accepted empirical and theoretical relationships in determining the elastic modulus for the clay strata. Therefore, RAI 02.05.04-17 is resolved and closed.

The applicant calculated the large-strain elastic modulus for the site-specific sands using an empirical method derived from a study performed on New England Sands and gravels, in addition to a shear wave velocity method, and averaged the results using a weighted formula that favored the shear wave velocity-derived value in the ratio of 2:1. The staff issued RAI 02.05.04-18, requesting the applicant to explain why the two methods used to determine the elastic modulus generally provide different results, and why the shear wave velocity-derived results are favored by 2:1 when computing an average elastic modulus value for use in predicting immediate settlements.

Similar to the RAI 02.05.04-17 response, in its to RAI 02.05.04-18, dated April 1, 2009, the applicant restated that the small strain shear wave velocity-derived elastic modulus is not affected by the large strains that accompany SPT sampling, and therefore represents the highest available stiffness. The applicant computed the empirically-based modulus values to accompany the velocity-based modulus values for sand strata using FSAR Equation 2.5S.4-13. FSAR Table 2.5.4-14, shows that the two sets of values are compatible. The applicant stated that the small strain modulus is the highest achievable stiffness because it is measured in situ at non-destructive strains, making it the benchmark of stiffness. The applicant based the weighting factor of 2:1 in favor of the velocity-derived results because they are considered to be most representative of the in situ conditions.

The staff reviewed FSAR Table 2.5S.4-14, and noted that the SPT empirically-based elastic modulus was typically less, in some cases substantially less, than the shear wave velocity-derived modulus. The staff concluded that this difference is possibly due to the fact that the SPT is a very rugged test not particularly sensitive to age-related cementation that would destroy the soil structure without accounting for added stiffness due to cementation, whereas the small strain velocity test would include the stiffness due to cementation. Therefore, the staff concluded that the applicant was conservative in averaging the large strain adjusted velocity-derived modulus values with the empirically-derived elastic modulus. Therefore, RAI 02.05.04-18 is resolved and closed.

The applicant evaluated the elastic modulus (E) for clay and coarse-grained soils using the relationships of Davie and Lewis (1988), which assume that overconsolidated clays and sands behave in an elastic or pseudo-elastic manner when loaded below their preconsolidation pressure. In the literature, this relationship is described as valid for applied loads of up to one-half of the preconsolidation pressure. To complete this evaluation, the staff needed to know whether the ratio of the STP-applied load to Beaumont Formation preconsolidation pressures in the various strata is the same as the ratio used to develop the relationship. The staff issued RAI 2.05.04-19, asking the applicant to compare the preconsolidation pressure in each clay stratum to the imposed stresses to the maximum depth of interest, and if loading is greater than half the preconsolidation pressure, to indicate why this relationship is still valid for computing immediate settlements at the STP site.

In its response to RAI 02.05.04-19, dated April 1, 2009 (ML090930717), the applicant included changes to the FSAR in conjunction with the RAI 02.05.04-13, Supplement 1 response. The applicant used the elastic modulus of the various soil strata to estimate settlements because the soils behave as overconsolidated. Furthermore, the applicant based the settlement estimates on the dewatered condition with the water table maintained 1.5 m (5 ft) below the bottom of the excavation, throughout the process of loading the foundation areas. The applicant indicated that even in the dewatered condition, the effective stresses in the soil strata only exceed the preconsolidation pressures to a small degree and in limited locations.

Where preconsolidation pressures are exceeded, the applicant uses consolidation test data. The applicant stated that when dewatering is no longer necessary, the water table will rebound and buoyancy on the foundation base will reduce the effective stresses in all soil strata to less than the preconsolidation stress, thus supporting the use of the elastic modulus to model the soil for settlement purposes. Finally, the applicant concluded that it was reasonable to use the elastic modulus in spite of the fact that the loading exceeds one-half of the pre-consolidation pressure at times during loading, because the modulus ratio of large strain elastic modulus to small strain elastic modulus computed using the stress based approach is similar to the modulus ratio calculated using the strain-based approach.

The staff reviewed the applicant's submittal and observed that the strain-based approach used for the clays and the stress-based approach for the sands result in modulus ratios of approximately 0.3 for either method. Because the two methods are essentially equivalent, the applicant does not need to compare the imposed stresses with the preconsolidation pressure in the clay strata, as requested in RAI 02.05.04-19. Therefore, RAI 02.05.04-19 is resolved and closed.

In FSAR Section 2.5.4.2, the applicant calculated the elastic modulus assuming strain in the range of 0.25 to 0.50 percent. The staff issued RAI 02.05.04-20, asking the applicant to indicate the level of strain in the sands and clays for which the elastic modulus relationship was used.

In its response to RAI 02.05.04-20, dated April 1, 2009 (ML090930717), the applicant proposed changes in the FSAR mark-up submitted in conjunction with the RAI 02.05.04-13, Supplement 1 response. The applicant described both a strain-based approach for determining the large strain elastic modulus in clays, as well as a stress-based approach that incorporates the factor of safety with respect to the ultimate stress in the sand strata for determining the large strain elastic modulus in sands. The applicant noted that the velocity-based modulus values for the clay strata could be determined from either approach, because both methods yield a similar modulus ratio of approximately 0.3. The applicant also concluded that for Stratum N Sand, Clay, and deeper, the incremental induced stress levels are lower; the factor of safety is higher; and a modulus ratio of 0.5 is appropriate.

The staff considered the applicant's comments and noted that using the strain- and stress-based approaches yield similar results. The staff also confirmed that the applicant had predicted total settlements on the order of 27.9 cm (11 in.) over the depth of influence, occurring mostly within a depth of double the minimum foundation width. The staff estimated the average one-dimensional axial strain under the center line of the reactor at approximately 0.24 percent, which is in the ballpark of the range of large strain between 0.25 and 0.50 percent the applicant assumes. Similarly, the staff concluded that the lower induced stress levels for Stratum N Sand, Clay, and deeper support the use of higher elastic modulus values derived from using a lower factor of safety in the stressed-based approach. Given these confirmations, the staff concluded that the relationship used to compute the elastic modulus values is adequately conservative. Accordingly, RAI 02.05.04-20 is resolved and closed.

The staff noted that the exploration data contain CPT soundings showing a high pore water pressure response to the piezocone in zones of silt or clay, which appear to correspond to the depths of Strata D and F. The staff issued RAI 02.05.04-23, asking the applicant to discuss how the high pore water pressure response measured in the overconsolidated clay soils is interpreted, and to justify the strength parameters for Strata D and F in light of the CPT high positive pore water pressure response.

In its response to RAI 02.05.04-23, dated July 20, 2009 (ML092030132), the applicant stated that the CPT measures the pore water pressure behind the cone tip. In heavily overconsolidated soils with an OCR greater than 10, the applicant expects the pore water pressure response to be zero or negative. However, for lightly overconsolidated soils such as Strata D and F, the applicant noted that a high pore water pressure does not reveal anything about the OCR other than it is not greater than 10. From available literature, the applicant demonstrates that the pore water pressure response of Strata D and F are typical of normally to lightly overconsolidated soils. The applicant used CPT test data to supplement and confirm the site-specific oedometer test data, which were the governing means for obtaining the OCR values. The applicant also estimated the CPT-derived shear strength using a conservative cone factor derived from a correlation with site-specific laboratory shear strength data. Finally, the applicant confirmed the CPT-derived shear strength predictions using shear strength results derived from conservative SPT correlations, as well as the site-specific laboratory shear strength results. The applicant concluded that the shear strength values obtained from the CPT data are conservative.

The staff considered the applicant’s explanation regarding the pore water pressure response, including the fact that the pore water is measured at the back of the cone tip, which suggests that high strains will occur and the pore water pressure response will be negative only for highly overconsolidated clays. The staff concluded that similar responses observed for lightly overconsolidated clays confirm this explanation. The staff further concluded that the applicant’s justification for how the shear strength data are derived from the cone penetration test data supports the view that the data were reliably and conservatively obtained. The staff also concluded that since the laboratory strength tests are biased toward lower end values due to the effects of sample disturbance and weaknesses built into the soil fabric, the cone factor used to backfigure shear strength from CPT tip resistance introduced conservatism into the calculation of the CPT-derived shear strength values. Therefore, the staff concluded that the high pore water pressure response was not and need not be considered in evaluating soil compressibility or strength properties. Therefore, RAI 02.05.04-23 is resolved and closed.

FSAR Subsection 2.5S.4.2.1.1, states that Strata J Clay, K Clay, L, and N Clay are treated as low plasticity index soils in determining their respective elastic modulus values. The staff observed that the plasticity index for the strata are typically greater than 30 and the percentage of sand is typically less than 25 percent. The staff issued RAI 02.05.04-25, asking the applicant to provide additional data to support the assumption of sand-like behavior and the use of a higher value of elastic modulus applicable to cohesive soils with a PI of less than 30.

In its response to RAI 02.05.04-25, dated August 10, 2009 (ML092250658), the applicant noted that the study by Bowles (1997) provided the applicant with a formula to calculate elastic modulus that is applicable to stiff to hard cohesive soils as well as to sandy soils. The applicant justified using this formula by proving that the clay soils are stiff to hard. The staff checked the reference and concluded that the equation is applicable to both sandy soils and stiff clays. Therefore, the staff concluded that the use of a higher elastic modulus is appropriate for the Beaumont clays with a PI greater than 30 due to their hard consistency. Therefore, RAI 02.05.04-25 is resolved and closed.

The applicant used equations from Bowles (1997) that relate elastic modulus to shear strength and the OCR. The equations contain a choice of multipliers to compute the $E_s$. Bowles (1997) further indicates that for overconsolidated soils subject to relief of overburden, $E_s$ may be much smaller due to heave of the subgrade. Because the applicant selected the mid-range values for the calculations, the staff issued RAI 02.05.04-26, requesting the applicant to explain how the reduction in $E_s$ due to heave is accounted for in the settlement predictions.

In its response to RAI 02.05.04-26, dated August 10, 2009, the applicant stated that considerable engineering judgment is required in predicting heave and the amount of recovered heave that will occur during settlement, as there are no reliable theories available for this purpose. The applicant compared the predicted settlements from STP, Units 3 and 4, to actual settlements measured at STP, Units 1 and 2, and considered the foundation size, shape, and bearing level. The applicant concluded that the predicted settlements and the actual settlements compared well, which gives the applicant confidence in the selection of the elastic modulus used in the settlement for STP, Units 3 and 4.

The staff concluded that the process the applicant used to select the elastic modulus was reasonable. The staff also agreed with the applicant that the settlement and heave calculations are inexact and engineering judgment is required. The staff concluded that the applicant has prudently selected modulus values to represent the range of stiffness of the soils for settlement predictions. Therefore, RAI 02.05.04-26 is resolved and closed.

**COL License Information Items 2.28, 2.29, and 2.30**

FSAR Subsection 2.5S.4.2, addresses COL License Information Items 2.28, 2.29, and 2.30, which require the applicant referencing the ABWR DCD to describe the field investigations performed at the site, the laboratory tests performed on samples collected at the site, and the subsurface conditions inferred from the results of the field and laboratory investigations. The applicant describes the field investigations, including CPT and SPT results, as well as provides the boring logs for the subsurface investigations. The applicant also described the laboratory tests performed including index property tests, strength and consolidation tests, and other physical property tests needed for the characterization of the subsurface materials and for input in stability analyses. Finally, the applicant used the results of these investigations, as presented in SER Table 2.5S.4-1 (FSAR Table 2.5S.4-16, “Summary of Average Geotechnical Engineering Parameters”), to interpret the subsurface conditions at the site, including which strata, if any, need to be removed before construction to ensure site stability. The staff reviewed the information in FSAR Subsection 2.5S.4.2 and concluded that the applicant has adequately characterized the subsurface materials at the STP site using the results of the field and laboratory investigations. The staff concluded that the applicant has sufficiently addressed COL License Information Items 2.28, 2.29, and 2.30.

**Staff Conclusions Regarding Subsurface Properties**

The staff reviewed STP COL FSAR Subsection 2.5S.4.2 and concluded that the applicant has provided sufficient information regarding the field and laboratory investigations, as well as the subsurface conditions to address COL License Information Items 2.28, 2.29, and 2.30. The staff concluded that the field and laboratory investigations and subsurface conditions at the site that the applicant describes in FSAR Subsection 2.5S.4.2 form an adequate basis for the determination of the properties of the subsurface materials at the site. FSAR Subsection 2.5S.4.2 also meets the requirements of 10 CFR Parts 50 and 100.

2.5S.4.4.3 Foundation Interfaces

FSAR Subsection 2.5.4.3, addresses ABWR DCD COL License Information Item 2.30, the results of the investigation of subsurface conditions with descriptions of soil and rock supporting the foundations. The information includes soil characteristics presented as profiles through the seismic Category I structures, which show generalized subsurface features beneath these structures.

The staff's review focused on the comparison of the subsurface materials with the proposed locations of foundations of all seismic Category I facilities. The staff cross-checked the profiles and plot plans in detail with the results of all subsurface investigations conducted at the site to ascertain that there has been sufficient exploration.

The staff noted that although the Radwaste building substructure and the UHS pump house structures are seismic Category I safety-related structures, FSAR Figure 2.5S.4-2, "Power Block Plan STP 3 & 4," Revision 0 does not show the borings located within the footprint of these structures, as specified in the ABWR DCD and RG 1.132. Therefore, the staff issued RAI 02.05.04-1, requesting the applicant to provide the static and dynamic soil data and related stability analyses for the Radwaste building and UHS structures.

In its response to RAI 02.05.04-1, dated October 21, 2008 (ML082970562), the applicant provided supplemental data derived from the third quarter of 2007 subsurface exploration, in which borings were drilled at both the Radwaste building location and the relocated UHS pump houses. The applicant's response included all of the additional borings and laboratory test results and analyses, including boring logs and SPT results from the subsequent Drilling and Testing Program for the relocated UHS sites. The applicant compared the stratigraphy observed in these borings to other site-wide derived results and found little or no difference. The applicant also compared engineering properties obtained from laboratory tests to site-wide data and found little difference in the strength or consolidation properties. Using the results of these borings and of laboratory tests, the applicant performed static bearing capacity and settlement analyses for the Radwaste building and the UHS pump houses.

The staff determined that the number and depths of borings at each major structure, including the STP, Unit 4, Radwaste building and the relocated UHS structures follow the guidance of RG 1.132 and 1.138. The staff concluded that the applicant has collected sufficient numbers of soil samples for laboratory testing and has performed adequate numbers of various soil tests to characterize and determine soil properties for engineering analyses. The staff further concluded that the applicant has provided sufficient data to characterize the foundation soils supporting the various structures in the STP, Unit 3 and 4, power block areas. Therefore, RAI 02.05.04-1 is resolved and closed.

FSAR Subsection 2.5S.4.3, also addresses COL License Information Item 2.30, which requires the applicant referencing the ABWR DCD to describe the subsurface conditions at the COL site. The applicant provided detailed figures illustrating the relationship between the subsurface materials and the foundations of structures at the STP site. The staff reviewed this information and, when considered together with the results of the field and laboratory investigations described in FSAR Subsection 2.5S.4.2, concluded that the applicant has provided sufficient information in FSAR Subsection 2.5S.4.3, to adequately address the characterization of the subsurface materials, as outlined in COL License Information Item 2.30.

The staff reviewed STP COL FSAR Subsection 2.5S.4.3, and concluded that the applicant has described the relationship between the subsurface materials and the foundations of structures at the STP site and has adequately addressed COL license Information Item 2.30. The staff conclude that the foundation interfaces, as described by the applicant in FSAR Subsection 2.5S.4.3, form an adequate basis for the characterization of the foundation interfaces at the site and meet the requirements of 10 CFR Parts 50 and 100.

2.5S.4.4.4 Geophysical Surveys

FSAR Subsection 2.5.4.4 addresses, in part, ABWR DCD COL License Information Item 2.34, the determination of dynamic soil properties of the site in terms of shear modulus and material damping as functions of shear strain, and the ABWR Tier 1 site requirement for shear wave velocity of 304 m/s (1,000 fps).

Although the ABWR DCD specifies a minimum shear wave velocity of 304 m/s (1,000 fps), FSAR Figures 2.5S.4-39 through 2.5S.4-44, show soil profiles with a shear wave velocity of less than 304 m/s (1,000 fps). The staff issued RAI 02.05.04-12, requesting the applicant to discuss the shear wave velocities for the site with respect to the ABWR DCD Tier 1 criteria.

In its response to RAI 02.05.04-12, dated September 10, 2008 (ML082560248), the applicant stated that since the measured shear wave velocities do not meet the minimum value of 304 m/s (1,000 fps) required in the ABWR DCD, a Tier 1 departure is being prepared for NRC's approval. The applicant will perform a site-specific, soil-structure interaction analysis to confirm that the standard plant seismic responses for the Reactor and Control Buildings bound the results of the site-specific analyses.

The departure from the DCD recommended shear wave velocity, which also affects the soil structure interaction analysis in FSAR Section 3.7.1. The staff's evaluation and resolution of RAI 02.05.04-12 are in Subsection 3.7.1.4 of this SER.

FSAR Subsection 2.5S.4.4, also addresses, in part, COL License Information Item 2.34, which requires the applicant to assess the response of site materials to dynamic loading. The staff reviewed the suspension P-S Velocity Logging and seismic CPTs that the applicant performed in order to determine the Vp and Vs of the soils underlying the site. The staff concluded that the performance of these surveys, when considered together with the evaluation and the application of the data obtained in the determination of the subsurface material response to dynamic loading in FSAR Subsection 2.5S.4.7, is sufficient to address COL License Information Item 2.34.

The staff reviewed FSAR Subsection 2.5S.4.4, and concluded that the applicant has performed a complete and thorough survey of the STP site using a variety of geophysical testing methods. Furthermore, the staff found that the applicant has provided a sufficient discussion of the geophysical survey and test methods to address COL License Information Item 2.34 of the ABWR DCD. The staff concluded that the geophysical tests and methods, as described by the applicant in FSAR Subsection 2.5S.4.4, form an adequate basis for the geophysical surveys of the site and meet the requirements of 10 CFR 100.23.

2.5S.4.4.5 Excavation and Backfill

FSAR Subsection 2.5S.4.5, addresses ABWR DCD COL License Information Items 2.31 and 2.39; the site-specific foundation conditions; the extent of all seismic Category I excavations, fills, and slopes; excavating and dewatering methods; the sources, quantities, and static and dynamic engineering properties of borrowed materials; compaction requirements; results of test fills; and fill properties.

**Excavation**

In FSAR Section 2.5S.4, the applicant describes the excavation plan for the STP site. However, the description does not include the stability analyses for the deep temporary excavations. The staff issued RAI 02.05.04-2, requesting the applicant to provide the final excavation plan, slope stability analyses, retaining wall design, and excavation monitoring plan.

In its response to RAI 02.05.04-2, dated October 1, 2008 (ML082770138), the applicant described the proposed excavation plan for STP, Units 3 and 4, in detail, including the limits and depths of the excavation, the proposed permanent reinforced concrete retaining wall, the temporary retaining structures, the proposed crane foundation, and the monitoring plan for the excavation. The applicant stated that at its lowest point, the excavation will be approximately 28.6 m (94 ft) below the proposed rough grade requiring the removal of approximately 2.6 million cubic meters (3.5 million cubic yards) of soil. The applicant plans to use conventional earth moving equipment and possibly conveyors to remove the soil from the deepest part of the excavation. The applicant also described the planned construction of side slopes, 1 vertical to 2 horizontal (1v:2h), with 6.096-m (20-ft) wide berms spaced at approximately 6.096 m (20 ft) intervals up the slope, making the final slopes effectively 1v:3h for excavation. It will not be practical in some areas of the excavation to lay back the slopes to a 1v:3h side slope due to site restrictions, so the applicant plans to use either steeper slopes of 1v:1.5h or retaining walls. The applicant will also place a permanent reinforced concrete wall to the east side of the STP, Units 3 and 4, Reactor and Turbine buildings to allow the placement of a heavy lift crane foundation. Finally, the applicant identified other temporary retaining structures as either tied-back steel sheet pile walls or soldier piles and timber lagging located between adjacent structures where safe slopes are not possible.

The applicant stated that the exposed portion of the reinforced concrete wall will be a maximum of 27.4 m (90 ft) high and tied back with anchors after excavation. The applicant presented the expected lateral pressures on the wall in the supplement accompanying the RAI response, as well as load capacity design charts for the proposed auger cast pile foundations. Behind the wall, the applicant plans to construct a permanent foundation for a heavy lift crane. Finally, the applicant noted that the crane loading will minimally affect loads on the reinforced concrete wall.

The applicant performed slope stability analyses using the GSTABL7 computer program that incorporates variable slopes, varied phreatic surface drawdown, and surcharge loads located on the berms. Using Bishop's method (Bishop, 1955) and conservative soil shear strength values, the applicant performed the analyses for circular arc failure surfaces and concluded that a ground water surface drawdown of 1.5 m (5 ft) below the excavation side-slopes is sufficient to produce a satisfactory factor of safety of at least 1.3. The applicant stated that additional soil borings and laboratory analysis are required to confirm the slope stability analysis. The applicant plans to perform these additional analyses using Janbu's method (Janbu, 1954) for non-circular surfaces for select cases.

The staff reviewed the slope stability analysis for the temporary excavation and concluded that the applicant has submitted sufficient information regarding the proposed excavation. The staff further concluded that the slope profile, variable ground water surface assumptions, material properties, and analytical procedures used to obtain the minimum factor of safety for the assumed critical case are acceptable.

The staff also found that the use of GSTABL7 to perform the slope stability analyses is acceptable. The staff further concluded that the use of the modified Bishop circular arc analysis procedure to run the analyses and the confirmatory use of Janbu's non-circular slide surface routine to check several slope configurations are also acceptable.

Although the applicant has performed stability analyses for the temporary excavation and assumes that friction and cohesion will be operative for the duration of the open excavation, the applicant did not address the potential for progressive failure or strength degradation. The applicant also did not justify the strength parameters used in the analyses. In order to ensure conservatism in the selection of the strength parameters, the staff issued supplemental RAI 02.05.04-24, requesting the applicant to: (1) discuss the operational shear strength of the stiff fissured Beaumont clay for the open excavation duration, (2) explain how this duration compares to the construction schedule for STP, Units 1 and 2, and (3) clarify whether there are any other long-term deep excavations in the Beaumont clay that would substantiate these assumptions.

In its response to RAI 02.05.04-24, dated August 10, 2009 (ML092250658), the applicant referred to research by Skempton (1964; 1970; and 1977), Mesri and Shahien (2003), and Gulla et al. (2006) showing that the overconsolidated soils tend to a fully softened state with a reduction of shear strength comparable to that of a normally consolidated state. The applicant considered progressive failure and implies that the assumption of a cohesion value of 14.3 kPa (300 psf) is representative of the fully softened state, noting that drained tests on soil samples from the strata under consideration had cohesion test results in the range of 47.8 to 110 kPa (1,000 to 2,300 psf). The applicant also used similar slopes for STP, Units 1 and 2, that performed satisfactorily for the four-year construction period, which is roughly the same duration that the STP, Units 3 and 4, excavations will be open. Based on this evaluation, the applicant concluded that a cohesion value of 14.3 kPa (300 psf) is a conservative parameter to use in the slope stability analysis.

Based on the range of drained shear strength values recorded for the Beaumont clay, and the experiential evidence cited for stability of slopes at STP, Units 1 and 2, the staff concurred with the applicant that the use of a cohesion value of 14.3 kPa (300 psf) is acceptable. The staff also considered available literature and concluded that the highly overconsolidated Beaumont clays will not have fully softened during the four-year construction period, and some portion of the cohesion will still be operational during the construction period. Therefore, RAI 02.05.04-24 is resolved and closed. The resolution of RAI 02.05.04-24 also closes RAI 02.05.04-2.

**Monitoring Program**

In FSAR Subsection 2.5S.4.5.4, the applicant describes the proposed settlement and heave monitoring at various stages of construction for major structures. The applicant will develop the monitoring program specifications during the detailed design phase. The staff issued RAI 02.05.04-4, requesting the applicant to submit the settlement and heave monitoring program. These plans are critical to ensure that the seismic Category I structures are not overstressed.

In its response to RAI 02.05.04-4, dated October 1, 2008 (ML082770138), the applicant provided a detailed schedule of instruments, instrument locations, and monitoring programs for the existing STP, Units 1 and 2, the main cooling reservoir, and the area between the existing structures and the construction limits for STP, Units 3 and 4. The applicant plans to use existing and new monuments, piezometers, extensometers, and slope inclinometers to monitor surface settlement and horizontal movement, changes in water table levels, settlement of strata at depth, and movement of slopes.

The applicant plans to commence monitoring three months before construction, which the applicant will accomplish either manually or remotely, depending on the instrument, with a frequency of readings during construction of twice per week or more, as dictated by conditions. The applicant also plans to determine a range of expected values for each point to which the acquired data will be compared. The applicant included a contingency plan with a graded course of action for data outside of the expected range of results.

The staff reviewed the instrumentation program and concluded that the applicant has selected existing instruments and plans for new instruments that should provide timely data to monitor settlement, heave, slope movement, and water table fluctuations at and between the existing STP, Units 1 and 2 and the main cooling reservoir. Furthermore, the staff concluded that starting the monitoring program three months in advance of construction is sufficient to develop the necessary baseline data, and the frequency of readings should provide sufficient data for monitoring structures and ground responses to construction operations. The staff also concluded that the plan to have expected ranges of responses for the various instruments will allow the applicant to adequately evaluate the acquired data and to implement contingency plans in cases where responses are outside of the expected range. Finally, the staff concluded that the applicant is well-positioned to monitor the existing site structures during and post-construction. Therefore, RAI 02.05.04-4 is resolved and closed.

**Backfill Specifications**

RG 1.206 states that the applicant should describe the sources and quantities of backfill materials, including the static and dynamic engineering properties of the materials. However, FSAR Table 3.0-11, “Backfill Under Seismic Category I Structures,” does not specify any inspections, tests, analyses, and acceptance criteria (ITAAC) to ensure that the properties of the backfill meet the site-specific assumptions. The table only commits to meet the minimum density values. The staff issued RAI 02.05.04-27 and RAI 02.05.04-31, requesting the applicant to describe how to ensure that the backfill meets or exceeds the design assumptions. The applicant also has to provide the assumed shear wave velocity, compressibility properties, and shear strength parameters for the backfill placed under seismic Category I structures.

In its response to RAI 02.05.04-27, dated August 10, 2009 (ML092250658), and RAI 02.05.04-31, dated October 12, 2009 (ML092890084), the applicant referred to its response to RAI 14.03.02-6, where the applicant stated its planto use quality control procedures, to verify key parameters of the backfill materials, are in FSAR Table 2.5S.4.5.3-1, "Quality Control Recommendations for Structural Fill." The applicant also proposed new ITAAC in FSAR Table 3.0-11 to require testing and verification of shear wave velocity as compared to the value used in design analyses, in addition to the ITAAC previously proposed for verifying backfill compaction. The applicant also stated that after identifying the source of backfill material to be placed under seismic Category I structures, it will test the materials to ensure that the backfill properties, such as compressibility and shear strength, are consistent with design inputs used in the analysis of these structures. The applicant also plans to characterize the backfill materials by key indicator parameters such as gradation, moisture content, Atterberg limits, and density that will be used for field quality control of the placed backfill. Finally, the applicant established the relationship between these key indicator parameters and the design input parameters to ensure that the backfill placed under seismic Category I structures meets or exceeds the requirements of the design analyses. The staff concurred with the applicant that the cited quality control procedures are sufficient to ensure that the backfill meets or exceeds the design assumptions during construction.

However, in order to quantify the static and dynamic properties of the granular backfill, the staff needed additional information and issued supplemental RAI 02.05.04-33. This RAI asked the applicant to describe the types and frequency of testing that will be performed to ensure that critical soil parameters such as strength, compressibility, shear modulus degradation, and damping ratio will bound the soil properties assumed in design for the range of backfill types that will be encountered in the placement of 1.6 million cubic meters (2.2 million cubic yards) of backfill. The staff specifically asked the applicant to: (1) specify the types and frequency of tests and (2) explain how the quality control program will ensure that the assumed soil parameters used in the site-specific design analyses are bounded by as-built backfill soil parameters.

In its response to RAI 02.05.04-33, dated January 21, 2010 (ML100250137), the applicant referred to the backfill ITAAC provided in response to RAI 14.03.02-6, and the testing plan described in response to RAI 02.05.04-31, to ensure backfill properties are consistent with the design inputs. The applicant plans to test each backfill source at the site to ensure that the design parameters are met. The applicant also added Table 2.5S.4.3-1 in Revision 3 of the COL FSAR for specifying the type and frequency of the tests. Furthermore, the applicant plans to evaluate the as-built soil parameters to ensure that the values are at least as good as those values assumed in the engineering analyses. In addition, the applicant plans to use a test fill pad to verify compaction equipment, the number of passes and other relevant data to achieve the specified compaction. The applicant will also develop an engineering report to confirm that the material, equipment and methods will produce acceptable and consistent results. The applicant proposed to revise FSAR Subsection 2.5S.4.5.3 and Table 2.5S.4.5.3-1, to include this information.

In its supplemental response to RAI 02.05.04-33, dated March 15, 2010 (ML100770389), the applicant proposed to incorporate the commitments in the initial response to RAI 02.05.04-33 into an ITAAC. The applicant will also revise the compaction specification to be consistent with the Quality Assurance Program requirements. The staff confirmed that the applicant revised the FSAR changes in Subsection 2.5S.4.5.3 and Table 2.5S.4.5.3-1 to include more detailed compaction specifications and this information was included in COL FSAR Revision 4. Therefore, this issue in RAI 02.05.04-33 is resolved and closed.

The staff reviewed the applicant's information on testing, frequency of testing that it plans to use in order to ascertain as-built material characteristics, including static and dynamic engineering properties. The applicant supplied information detailing the proposed test fill plan, and provided a backfill ITAAC as a tool to confirm that the engineering properties of the backfill materials are bounded by the assumed values. However, the staff concluded that certain testing frequencies were insufficient and the backfill ITAAC lacked specificity.

The staff issued supplemental RAI 02.05.04-34, asking the applicant to: (1) verify the frequency of in-place density testing for backfill supporting seismic Category I structures, (2) justify the proposed frequency of moisture-density testing to ensure that the material property changes are recognized quickly, (3) provide the laboratory test results and analyses or provide and justify alternate criteria to verify the assumed parameters from the engineering analyses are met, and (4) provide the assumed shear modulus degradation and damping ratio versus strain relationships.

In its response to RAI 02.05.04-34, dated April 1, 2010 (ML100980067), the applicant addressed each question separately. To address the verification of test frequency for in-place density testing, the applicant refers to the supplemental response to RAI 02.05.04-33, which increases the minimum density testing frequency to one test for every 200 cubic yards of backfill placed at the site, which is consistent with NQA-1.

As its justification of the proposed frequency of moisture density testing, the applicant again refers to the supplemental response to RAI 02.05.04-33, which increases the minimum testing frequency to one test for every 10 field density tests to be consistent with NQA-1. The applicant also refers to the supplemental response to RAI 02.05.04-33, which adds the backfill ITAAC to include a design requirement that the engineering properties under seismic Category I structures bound the values used in the site-specific design analyses. Finally, the applicant refers to the revision to FSAR Subsection 2.5S.4.5.3, which details the quality control methods and testing. In addition, the applicant proposed an ITAAC requiring backfill properties to be consistent with the assumptions made during the course of the static and dynamic engineering analyses. The ITAAC will confirm that the relationships assumed for shear modulus degradation and damping ratio versus shear strain in the engineering analyses bound the dynamic properties of the backfill. The applicant also updated FSAR Subsection 2.5S.4.7.3.7, to refer to the revised backfill ITAAC.

The staff reviewed the proposed revisions to the FSAR and found them acceptable.

The staff reviewed the applicant's response to RAI 02.05.04-34, including the referenced supplemental response to RAI 02.05.04-33, wherein the frequency of in-place density testing and the frequency of modified Proctor compaction testing was increased. The staff concluded that this increased frequency of modified Proctor compaction testing is reasonable because it is frequent enough to monitor changes in material type. Given the newly proposed criteria of in situ density testing frequency at the rate of 1 test per 200 cubic yards of fill placed, the staff concluded that good controls will be in place to monitor fill compaction and uniformity of moisture content. The staff further concluded that the increased rate in modified Proctor compaction testing, 1 test for every 10 in situ density tests, will ensure that any material changes will be quickly recognized and that the in-place density results will be compared to the proper compaction curves. However, the staff concluded that the applicant's response still lacked the specific dynamic soil property relationships for shear modulus and damping that was assumed in the dynamic engineering analyses. These values are needed to compare with values measured during construction.

The staff reviewed both the shear wave velocity and settlement ITAAC and recognized that both ITAACs lacked specific acceptance criteria. The staff issued supplemental RAI 02.05.04-36, requesting that the applicant update the proposed ITAAC to reflect demonstrations that the assumption in the safety analyses are verified and consistent with the requirements of 10 CFR 100.23.

In its response to RAI 02.05.04-36, dated June 3, 2010 (ML101590397), the applicant addressed each ITAAC on backfill properties, shear wave velocity, engineering properties of backfill, and settlement, separately as shown in SER Table 2.5S.4-7, "Laboratory Testing Summary," (COL application Part 9, Section 3, Table 3.0-11, "Backfill Under Seismic Category I Structures"), and in SER Table 2.5S.4-10 for settlement (COL application Part 9, Section 3, Table 3.0-15, "Settlement"), respectively. The staff's evaluation of the settlement ITAAC is presented in Subsection 2.5S.4.4.10 of this SER.

**Table 2.5S.4-7 Backfill Under Seismic Category I Structures (COL Application Part 9, Table 3.0-11)**

| Design Requirement | Inspections, Tests and Analyses | Acceptance Criteria |
|---|---|---|
| 1. Backfill material under seismic Category I structures is installed to meet a minimum of 95 percent of the Modified Proctor density. | 1. Testing will be performed during placement of the backfill materials. | 1. A report exists that concludes the installed backfill material under seismic Category I structures meets a minimum of 95 percent of the Modified Proctor density. |
| 2. The shear wave velocity of backfill under seismic Category I structures meets the value used in the site-specific design analyses. | 2. Field measurements and analyses of shear wave velocity in backfill will be performed when backfill placement is at approximately the elevations corresponding to: (1) half the backfill thickness to be placed below the foundation level, (2) the foundation depth (i.e., base of concrete fill), and (3) the finish grade around the structure. | 2. An engineering report exists that concludes that the shear wave velocity within the backfill material placed under seismic Category I structures at their foundation depth and below is greater than or equal to 600 feet/second for the RSW Tunnels and Diesel Generator Fuel Oil Storage Vaults and 470 feet/second for the Diesel Generator Fuel Oil Storage Vault Tunnels. |

| Design Requirement | Inspections, Tests and Analyses | Acceptance Criteria |
|---|---|---|
| 3. The engineering properties of backfill under seismic Category I structures bound the values used in the site-specific design analyses. | 3. Laboratory tests, field measurements and analyses of engineering properties of the backfill will be performed. | 3. An engineering report exists that concludes that the engineering properties of backfill under seismic Category I structures (unit weight, phi angle, shear strength, shear modulus, shear modulus degradation and damping ratio) meet the values used in the site-specific design analyses. |

For the shear wave velocity, the applicant proposed an additional ITAAC to address the shear wave velocity requirements and plans to revise the FSAR to include this change. With respect to the engineering properties of the backfill, the applicant stated that the ITAAC addressing the backfill beneath seismic Category I structures will confirm that the engineering properties of the laboratory analyses meet the site-specific design analysis values. The applicant also plans to revise Table 3.0-11, to include the compaction, shear wave velocity and engineering properties, as well as provide additional criteria for the engineering properties of the backfill. Furthermore, the staff noted that the revised shear wave velocity ITAAC now contains the specific values of shear wave velocity assumed in design that must be met or exceeded in the field to be acceptable. The staff therefore concluded that the shear wave velocity ITAAC is acceptable to verify that the actual shear wave velocity values equal or exceed the values assumed in the design analysis.

The applicant has provided in its response to RAI 02.05.04-36, all the specific data requested by the staff in RAI 02.05.04-27, RAI 02.05.04-31, RAI 02.05.04-33 and RAI 02.05.04-34, including the on-site backfill testing plan, types of tests, frequency of testing and specific assumed backfill material properties. The staff concluded that the applicant's quality assurance plan for backfill placement is sufficient to ensure that the soil properties of compacted in place fill underlying seismic Category 1 structures, diesel generator fuel oil storage vaults and RSW tunnels will be adequate to provide static and dynamic stability to these structures. The staff noted that these are light structures exhibiting high factors of safety and large margins with respect to the static and dynamic demand. The staff concluded that this information is sufficient to resolve RAIs 02.05.04-27, 02.05.04-31, 02.05.04-33, 02.05.04-34, and 02.05.04-36. The staff also confirmed that the applicant's revised FSAR changes in Subsections 2.5S.4.5.3 and 2.5S.4.7.3.7 to include details of the quality control methods and testing and the revised backfill ITAAC are incorporated into Revision 4 of the COL application. Therefore, RAI 02.05.04-36 and its related RAIs (i.e., 02.05.04-27, -31, -33, and -34) are resolved and closed.

Although the applicant commits to updating the FSAR with the types of tests, frequency of testing and specific material properties that are to be measured for comparison with field values, the Backfill ITAAC itself must contain the types of tests and frequency of testing to be performed in the field to verify as-built properties bound the assumed engineering properties. Because, Item 3 of Table 3.0-11, does not contain the types of tests and frequency of testing required in the ITAAC, the staff issued RAI 02.05.04-37, requesting the applicant to provide the tests to be performed, as well as the testing frequency that will be followed in the ITAAC. This RAI was tracked as Open Item 02.05.04-37, in the SER with open items.

In its response to RAI 02.05.04-37, dated August 10, 2010 (ML100980067), the applicant modified Item 3 in Table 3.0-11 to include the tests to be performed and the testing frequency that will be followed in the ITAAC.

The staff reviewed the applicant's response and found that the modifications to Table 3.0-11, are sufficient. The structures to be placed on engineered fill have high margins of safety against static and dynamic bearing capacity failure, which gives reasonable assurance that for a range of engineering properties, the performance of the structure under static and dynamic loading will be satisfactory. Factors of safety for structures supported on backfill for the static case exceed 89, and factors of safety under dynamic loading exceed 34. FSAR Tables 2.5S.4-41B, "Bearing Capacity of Foundation," and 2.5S.4-41C, "Bearing Capacity of Foundations under Dynamic or Transient Loading," present all bearing capacity factors of safety for the static and dynamic cases, respectively. It should be noted that a factor of safety of 3 and 1.5 have been accepted for static and dynamic cases, respectively, for nuclear design. Because the density and shear wave velocity requirements are met and granular soils are used, the engineering properties of the fill will meet or exceed static and dynamic performance criteria. Based on a review of the applicant's conservative assumptions for static and dynamic material properties, and the commitment to reanalyze to ensure safety, the staff has reasonable assurance that backfill placement under the seismic Category 1 structures can have a range of values and still achieve satisfactory performance. The staff therefore concluded that it is reasonable to require the applicant to perform certain tests at specified frequencies on the prospective borrow, as detailed in the engineering properties ITAAC, and to compare the results of those tests against the assumed values provided in the COL application. For those cases where the prospective borrow demonstrates values divergent from the assumed values, the applicant has the option to go to other borrow sites or to reanalyze using the values of the proposed borrow to ensure satisfactory performance, before using these materials in the structural fill. If the engineering properties of those materials predict satisfactory performance, then those materials would be acceptable for fill placement. The staff concluded that meeting this ITAAC will result in an engineered fill that meets the static and dynamic performance criteria, and together with the high factors of safety determined for the assumed values, the staff closed RAI 02.05.04-37. All RAIs related to backfill are therefore considered closed.

Revision 3 of the STP COL application indicates that there is concrete backfill below all of the seismic Category I structures ranging from 0.61 to 3.05 m (2 to 10 ft) thick. However, the staff was unable to locate the specifications or placement methods for the concrete backfill in FSAR Subsection 2.5.4.5. The staff issued supplemental RAI 02.05.04-32, requesting the applicant to provide concrete specifications and concrete placement methods for the concrete backfill proposed as backfill below the seismic Category I structures.

In its response to RAI 02.05.04-32, dated December 21, 2009 (ML093580191), the applicant provided the necessary information and plans to design the concrete for the proposed backfill to have an unconfined compressive strength of 20.6 MPa (3,000 psi) at 28 days. The applicant will also test, inspect and place the concrete in accordance with ACI 349, "Code Requirements for Nuclear Safety-Related Concrete Structures," and other applicable requirements. The staff reviewed the applicant's response and concluded that based on the unconfined compressive strength at 28 days and the described plans to tests, inspect, and place the concrete in accordance with applicable industry standards, the applicant's response is acceptable. Therefore, RAI 02.05.04-32 is resolved and closed.

*COL License Information Items 2.31 and 2.39*

FSAR Subsection 2.5S.4.5, also addresses COL License Information Items 2.31 and 2.39, requiring the COL applicant referencing the ABWR DCD to describe the excavation and backfill for foundation construction and the subsurface monitoring plans. The applicant described excavation and foundation monitoring plans that sufficiently address the COL license information items. The applicant also discussed the backfill criteria and specifications. In response to supplemental RAI 02.05.04-33, and subsequent supplemental RAIs 02.05.04-34, 02.05.04-36, and 02.05.04-37, the applicant provided the information to ensure that backfill placement results in compacted soil properties that are bounded by design assumptions. The staff reviewed the response to these RAIs and concluded that the applicant has a satisfactory plan to ensure that the backfill meets the specifications and material property requirements assumed in the design phase. The staff's conclusion that the information submitted by the applicant is adequate to characterize the backfill and demonstrate conformance with the established criteria and specifications is based on the following:

- The applicant's plan to perform a test fill program to determine best practices and equipment for backfill placement in accordance with specifications to meet design assumptions;

- The applicant's plan to perform laboratory tests on samples from proposed borrow areas to locate suitable material;

- The applicant's plan to perform appropriate tests on stockpiled borrow to ensure it is satisfactory prior to placement; and

- The applicant's plan to perform testing to ensure that the density and moisture content are within specification limits after compaction.

Accordingly, the staff concluded that the applicant has provided adequate information to satisfy COL License Information Item 2.31, with respect to the backfill criteria and specifications. In addition, the staff concluded that the applicant has provided adequate information to satisfy COL License Information Item 2.39, with respect to subsurface instrumentation and monitoring.

**Conclusions Regarding Excavation and Backfill**

The staff reviewed STP COL FSAR Subsection 2.5S.4.5, and concluded that the applicant has developed and described a complete excavation and backfilling plan for the STP site, including the extent of the excavations. The staff concluded that the applicant's discussion of the excavation plans, extent, and methods are sufficient to address COL License Information Items 2.31 and 2.39, regarding the excavation of the foundation construction and subsurface instrumentation, respectively. Based on the response to RAI 02.05.04-33 and supplemental RAIs 02.05.04-34, 02.05.04-36, and 02.05.04-37, the staff concluded that the applicant has provided adequate information to address COL License Information Item 2.31, with respect to the backfill criteria and specifications.

### 2.5S.4.4.6 Ground Water Conditions

FSAR Subsection 2.5S.4.6.2, indicates that the applicant will use shallow well points and deep wells to draw the water table below the slopes and the bottom of the excavation. The applicant also presents plans to develop a dewatering plan as part of the detailed design. The staff issued RAI 02.05.04-3, requesting the applicant to provide the dewatering plan and the dewatering monitoring plan.

In its response to RAI 02.05.04-3, dated October 1, 2008 (ML82770138), the applicant provided the detailed dewatering and monitoring plan, including a description of the types of deep wells and well points to dewater the excavation, the general location of the deep wells, calculations of the anticipated pumping rates for the required drawdown, and a detailed description of the slurry wall that will surround the excavation. The applicant also described the various monitoring instruments that will monitor the piezometric levels in various strata, the horizontal and vertical movement of slopes and retaining structures, and the heave at the bottom of the excavation during the excavation stage of construction.

The applicant plans to key the 0.92 to 1.5 m (3 to 5 ft) thick slurry wall into the Stratum J clay at a depth of approximately 38.1 m (125 ft), which positions the wall approximately 9.2 m (30 ft) from the edge of the excavation and continuously around the perimeter of the excavation. The applicant noted that the minimum permeability of the slurry wall will be $1 \times 10^{-6}$ cm/sec to effectively cut off ground water movement into the excavation area. The applicant also plans to monitor the piezometric levels both inside and outside of the slurry wall to ensure that ground water conditions outside of the slurry wall are minimally affected by pumping in the deep well inside of the slurry wall. Finally, the applicant provided threshold values for inclinometers and piezometers to assist in the evaluation of the acquired data.

The staff reviewed the proposed method for constructing the slurry wall and concluded that it will effectively minimize the flow into the excavation area and will sufficiently minimize the effect of dewatering on existing facilities outside the limits of construction. The staff further concluded that the use of deep wells and well points is sufficient to draw the water within the excavation limits down to at least 0.92 to 1.5 m (3 to 5 ft) below the slopes and the bottom of the excavation. The staff also found that the applicant's plans will effectively monitor the phreatic surface, the piezometric pressures in strata inside and outside of the slurry wall, and the slopes and bottom heave during excavation Finally, the staff concluded that the applicant has properly evaluated all of the important aspects of the excavation and dewatering requirements, including considerations of the construction requirements necessary to successfully dewater the site without affecting the surrounding structures. Therefore, RAI 02.05.04-3 is resolved and closed.

The staff reviewed the supplemental dewatering information, which indicates that the drawdown will be a minimum of 1 m (3 ft) below the side slopes, although stability analyses indicate that to achieve an acceptable factor of safety, a drawdown to a depth of 1.5 m (5 ft) is necessary. The staff issued RAI 02.05.04-16, asking the applicant to coordinate the information in the supplemental dewatering plan and temporary excavation slope stability analyses.

In its response to RAI 02.05.04-16, dated April 1, 2009 (ML090930717), the applicant clarified the dewatering information in Dewatering Plan Revision D, Section 2.1, of the supplemental information submitted as part of the applicant's response to RAI 02.05.04-3. The applicant stated that the dewatering system will produce a minimum drawdown of 1 m (3 ft) below the bottom of the excavation and a minimum drawdown of 1.5 m (5 ft) below the slopes.

The staff reviewed this RAI response and concluded that lowering the ground water 1 m (3 ft) below the bottom of the excavation is acceptable for maintaining a stable subgrade, and the drawdown of 1.5 m (5 ft) below the slopes will achieve the minimum factor of safety of 1.3 for the excavation slopes, as determined by the slope stability analyses. Therefore, RAI 02.05.04-16 is resolved and closed.

FSAR Subsection 2.5S.4.6, also addresses COL License Information Item 2.32, requiring the applicant referencing the ABWR DCD to address the ground water conditions at the COL site. Although most ground water conditions are addressed in FSAR Section 2.4, the staff concluded that the applicant's description of the drawdown effects and dewatering plan for the STP site included in FSAR Subsection 2.5S.4.6, adequately addresses the geotechnical engineering aspects of COL License Information Item 2.32.

The staff reviewed STP COL FSAR Subsection 2.5S.4.6, and concluded that the applicant has accurately assessed the ground water conditions at the site from a geotechnical engineering standpoint, particularly the interaction of the ground water with the excavation, backfill, and structural foundations.

The staff further concludes that the applicant's information sufficiently addresses COL License Information Item 2.32, regarding the ground water conditions at the site. The staff also concluded that applicant's description in FSAR Subsection 2.5S.4.6, of the relationship between ground water, excavation, backfill, and the foundations of structures at the STP site forms an adequate basis for assessing the ground water conditions at the site and meets the requirements of 10 CFR Parts 50 and 100.

2.5S.4.4.7 Response of Soil and Rock to Dynamic Loading

The staff noted at the time of the original submission of the STP COL application that although the applicant has committed to perform additional RCTS tests (COM 2.5S-1), only a limited number of shear modulus and damping curves would be available. Therefore, the staff issued RAI 02.05.02-17, asking the applicant to provide the additional test results to ensure that the literature-based EPRI curves used to calculate the GMRS are representative of the site-specific conditions.

In its response to RAI 02.05.02-17, dated July 2, 2008 (ML081890239), the applicant revised the COL application to include additional RCTS testing in FSAR Subsection 2.5.4.7, which increases the total number of RCTS tests from five to sixteen. The applicant also presents in FSAR Figures 2.5S.4-62 through 2.5S.4-68, site-specific shear modulus and damping ratio curves developed for each stratum down to a depth of approximately 182.8 m (600 ft). The applicant demonstrates that the selected shear modulus and damping ratio versus strain relationships from the literature closely match the site-specific developed curves. Because the literature-based curves were carried out at higher strain levels than the site-specific curves, the applicant used the literature-based curves to calculate the GMRS. SER Figures 2.5S.4-7 and 2.5S.4-8 (FSAR Figures 2.5S.4-62, "Shear Modulus Degradation Based on RCTS Testing – All Sand Samples," and 2.5S.4-66, "Damping Curve Measurements Based on RCTS Testing - Sand Samples"), compare the site-specific shear modulus and damping ratio curves performed on all sand stratum to the EPRI curves, respectively.

The applicant selected the generic shear modulus curves for cohesionless soil based on the stratum depths shown in SER Figure 2.5S.4-7. Similarly, the applicant selected generic shear modulus degradation curves for cohesive soil strata based on the PIs of the strata. SER Figures 2.5S.4-9 and 2.5S.4-10 (FSAR Figures 2.5S.4-64, "Shear Modulus Degradation Based on RCTS Testing - High PI Clay Sample," and 2.5S.4-65, "Shear Modulus Degradation Based on RCTS Testing - Low PI Clay Sample"), compare the high- and low-plasticity cohesive soils to the literature-based curves, respectively.

The applicant developed generic damping ratio curves for cohesionless and cohesive soil strata in a similar manner. Given that the RCTS test results cover a wide range of confining stresses between 689 and more than 2,757 kPa (100 to more than 400 psi) and frequencies ranging from 0.5 to more than 80 Hz, some spread in the results is expected.

To determine the GMRS for the sand and clay strata at the STP site, the applicant concluded that there is good agreement between the site-specific RCTS test results and the literature-based curves and between selected specific literature-based shear modulus degradation curves and damping ratio curves. For sands located at depths greater than 30.48 m (100 ft), the applicant selected the EPRI shear modulus degradation curve for depths between 152.4 and 304.8 m (500 and 1,000 ft). For sands located at depths less than 30.48 m (100 ft), the applicant used the EPRI curve for depths of 106.6 to 152.4 m (350 to 500 ft). For clays with a PI greater than 30, the applicant used the Vucetic and Dobry (1991) curve for a PI of 100. For clays with a PI less than 30, the applicant used the Vucetic and Dobry (1991) curve for a PI of 50. Finally, for silt, the applicant selected an EPRI curve for a PI of 50.

With respect to the damping ratio, the applicant used the EPRI curve for depths that varied from 152.4 to 304.8 m (500 to 1,000 ft) for sands, and the Vucetic and Dobry (1991) curve for a PI of 200 for clays with a PI of greater than 30. For low PI clay and silt samples, the applicant used the Vucetic and Dobry (1991) curve for a PI of 200 up to strains of 0.005 percent and the EPRI-interpolated PI of 60 curve for strains above 0.05 percent. The applicant also referred to FSAR Subsection 2.5S.4.5, for structural fill requirements.

The staff reviewed the applicant's RCTS results and the comparisons to the literature-based curves presented in FSAR Figures 2.5S.4-62 through 2.5S.4-68. Except for the deep cohesionless soils in FSAR Figure 2.5S.4-63, "Shear Modulus Degradation Based on RCTS Testing - Deep Sand Samples," the staff agreed with the applicant that the site-specific curves are bounded by the literature-based curves and selected the literature-based curves that best define the trend of the site-specific curves for use in computing the GMRS with one exception: the deep cohesionless soils in SER Figure 2.5S.4-1 display a stiffer response than the EPRI curves. However, considering that the EPRI curve selected to represent the deep sand strata is not significantly different from the site-specific curves out to a strain of 0.1 percent, and the logarithmic mean maximum strain profiles in SER Figure 2.5S.4-11 (FSAR Figure 2.5S.2-47, "Logarithmic Mean Maximum Strain Profiles"), are typically significantly less than 0.1 percent strain, the staff concluded that the selected literature-based curves track well with the site-specific curves and are appropriate for determining the GMRS. Finally, the staff concluded that the characterization of the dynamic properties of the in situ materials is complete, therefore, RAI 02.05.02-17 is resolved and closed.

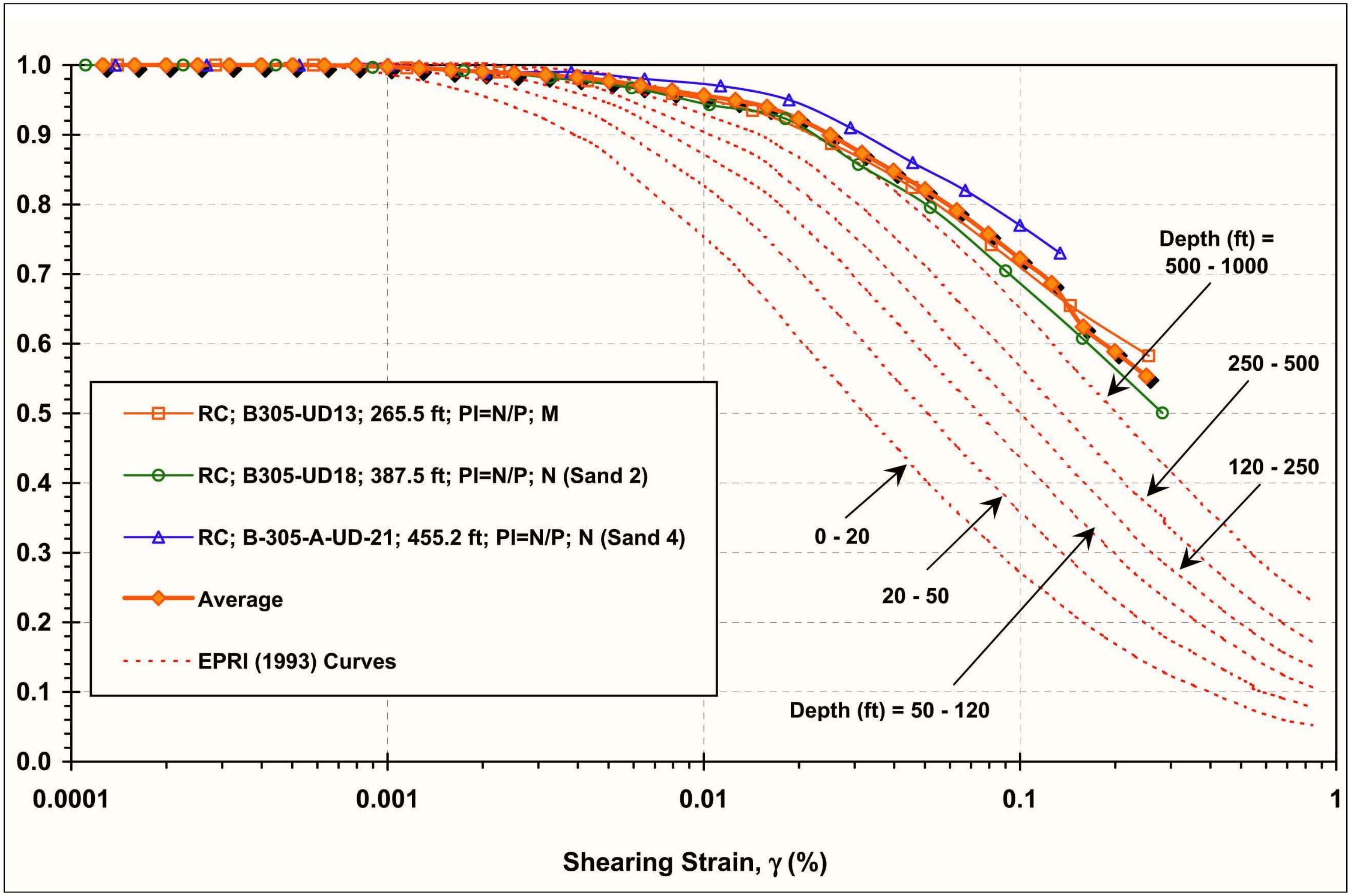


**Figure 2.5S.4-7 Site-Specific Sand Shear Modulus Curves Based on RCTS Testing (FSAR Figure 2.5S.4-62)**

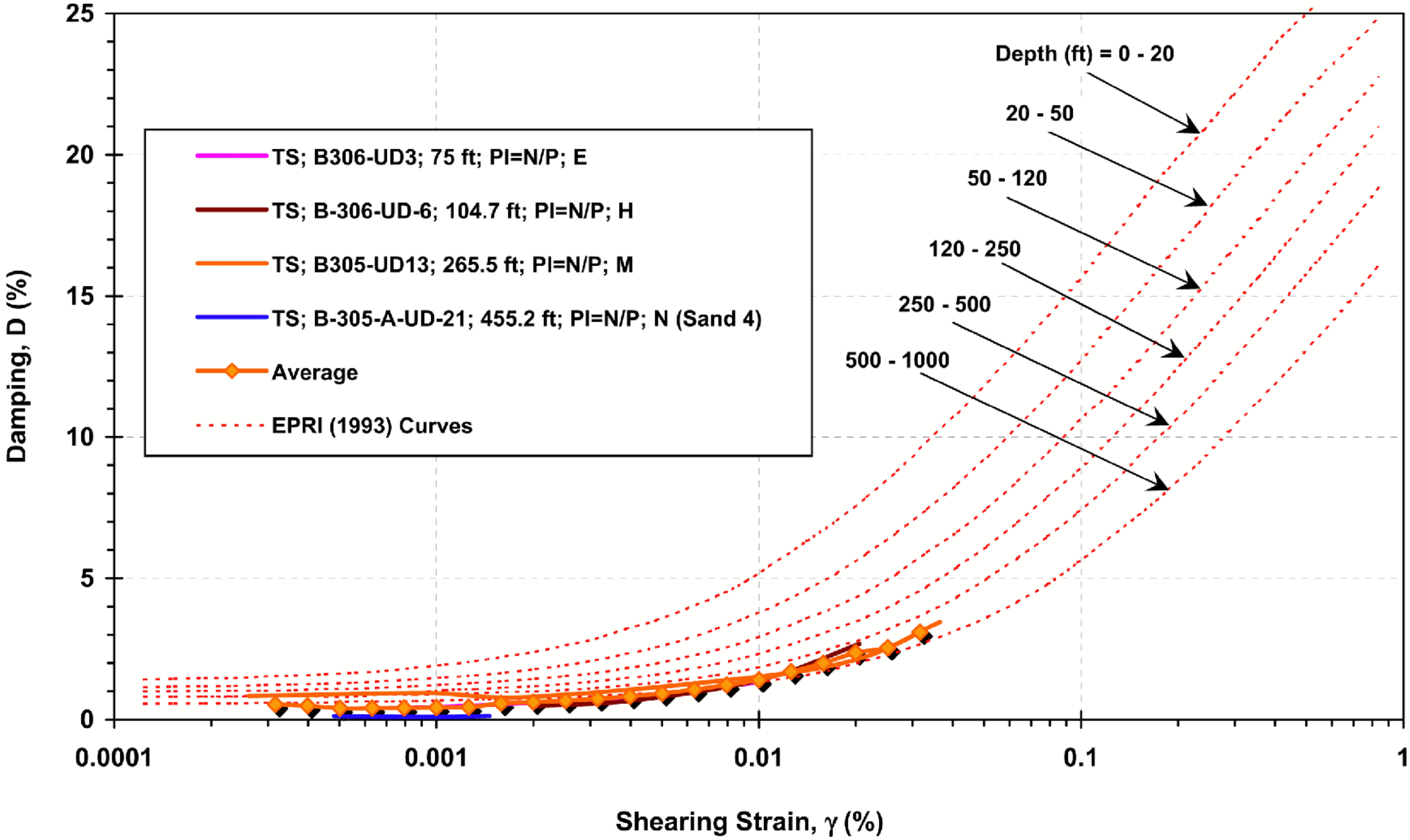


**Figure 2.5S.4-8 Site-Specific Sand Damping Ratio Curves Based on RCTS Testing (FSAR Figure 2.5S.4-66)**

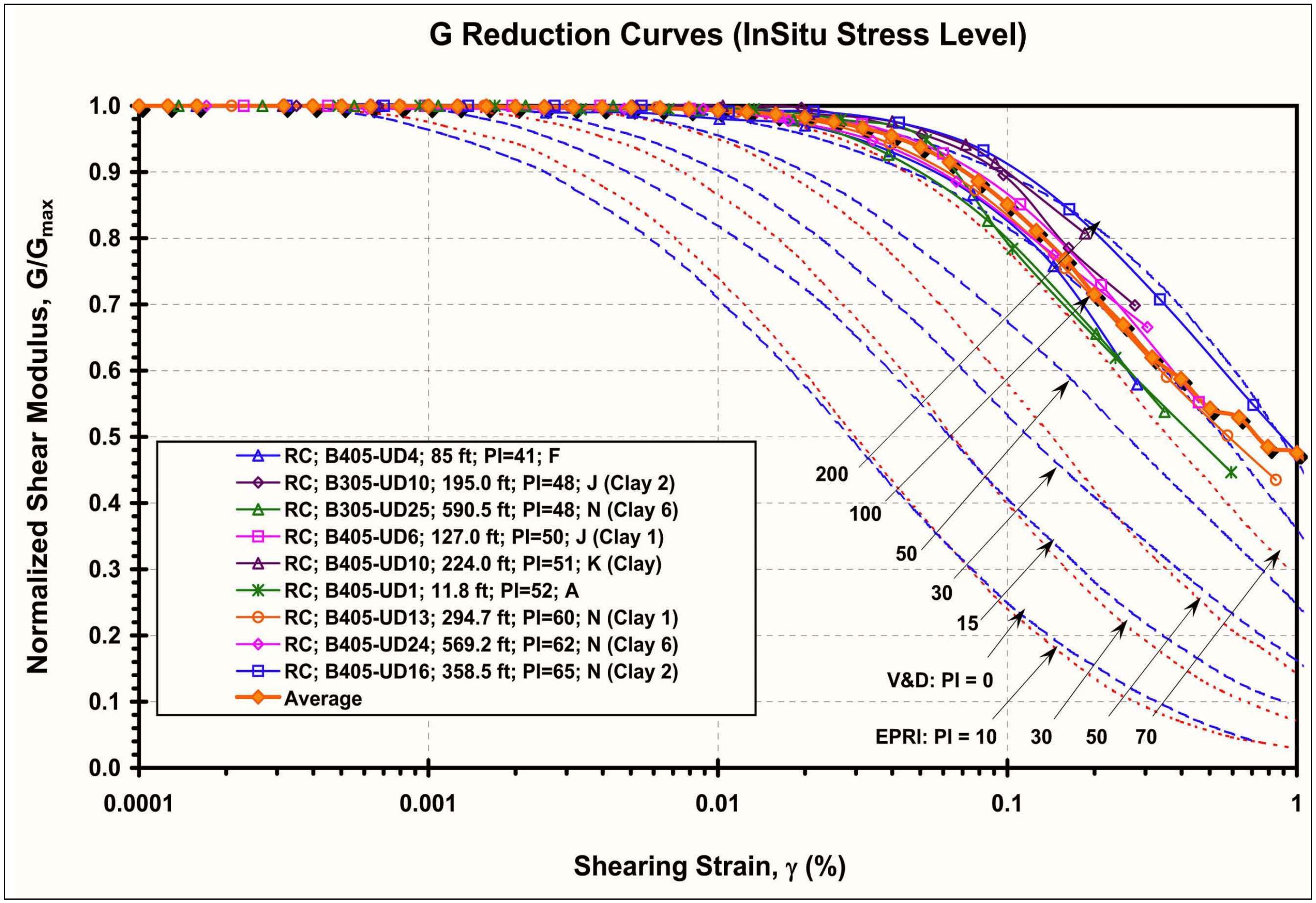


**Figure 2.5S.4-9 Shear Modulus Curves for High Plasticity Cohesive Soils Based on RCTS Testing (FSAR Figure 2.5S.4-64)**

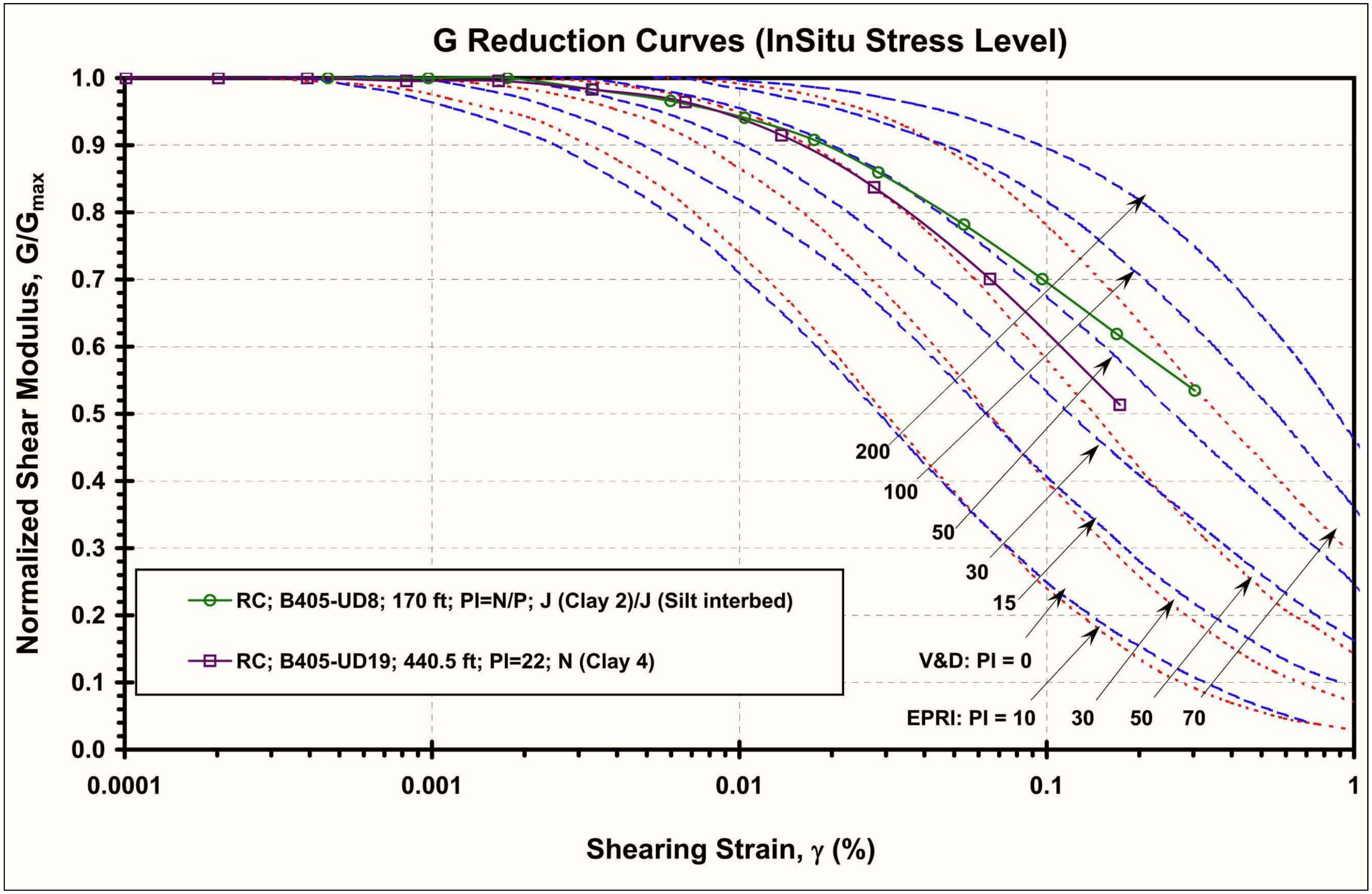


Figure 2.5S.4-10 Shear Modulus Curves for Low Plasticity Cohesive Soils Based on RCTS Testing (FSAR Figure 2.5S.4-65)

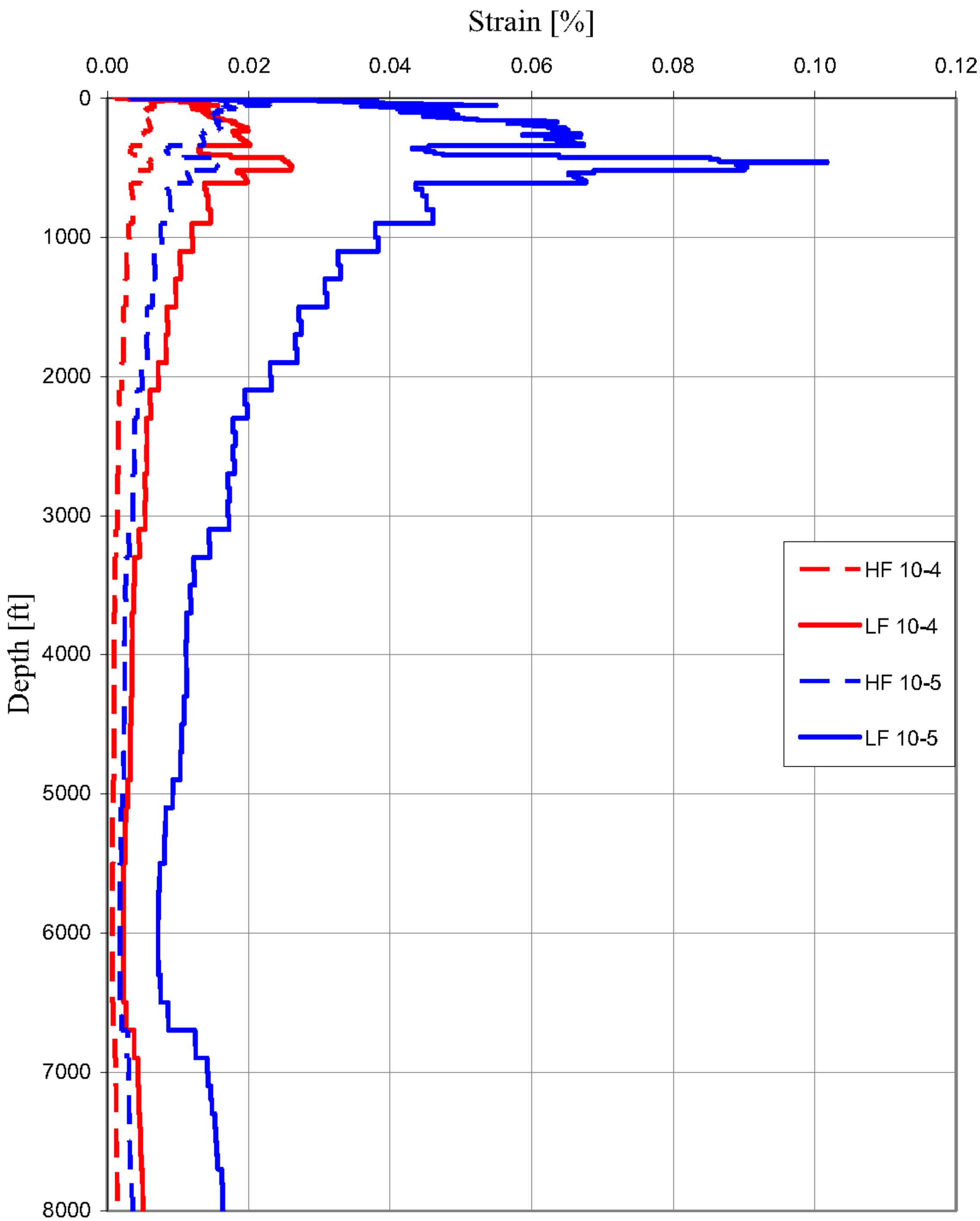


**Figure 2.5S.4-11 Logarithmic Mean Maximum Strain Profile (FSAR Figure 2.5S.2-47)**

FSAR Subsection 2.5S.4.7, also addresses COL License Information Item 2.34 requiring the COL applicant referencing the ABWR DCD to address the response of the subsurface materials to dynamic loading through the determination of shear modulus and damping. The applicant used the results of RCTS testing to determine the shear modulus and damping ratio curves, which were compared to literature-developed curves. The staff reviewed this information and determined that it adequately addresses the reporting requirements of COL License Information Item 2.34.

The staff reviewed FSAR Subsection 2.5S.4.7, and concluded that the applicant has characterized the dynamic properties at the STP site and has completely addressed the response of soil and rock to dynamic loading, thus satisfying COL License Information Item 2.34. The staff concluded that the applicant's characterization in FSAR Subsection 2.5S.4.7, of the dynamic properties of the subsurface materials forms an adequate basis for assessing the response of soil and rock to dynamic loading at the site and meets the requirements of 10 CFR Parts 50 and 100

2.5S.4.4.8 Liquefaction Potential

Although the ABWR DCD states that no liquefaction should occur within the STP site, FSAR Tables 2.5S.4-34A, "Summary of RCTS Laboratory Test Results"; 2.5S.4-34B"G/Gmax vs. Strain Based on RCTS Results"; 2.5S.4-34C "Damping Ratio vs. Strain Based on RCTS Results"; and 2.5S.4-35, "Summary of Liquefaction Potential FOS Values <1.10; SPT Method"; show points of liquefaction potential within subsurface strata determined from SPT and CPT results. The staff issued RAI 02.05.04-5, requesting the applicant to provide a graphic interpretation of the extent of the liquefiable zones and to justify the potential for liquefaction, with respect to the ABWR DCD requirement.

In its response to RAI 02.05.04-5, dated July 9, 2008 (ML081960070), the applicant concluded that although FSAR Subsection 2.5S.4.8, identifies a small number of sampled points with a factor of safety against liquefaction of less than 1.1, the liquefaction potential was small because of: (1) the overwhelming numbers of data points that were not liquefiable, (2) the planned removal of the liquefiable zones during excavation, (3) the lack of structures planned in a liquefiable zone, or (4) the fact that the stronger materials surround limited liquefiable zones. In the one instance where limited areas of potentially liquefiable soils underlay structure foundations at shallow depths, the mat foundations are large enough to bridge those isolated zones. The applicant also stated that a graphical presentation of liquefiable zones was not possible because there were no liquefaction zones.

The staff noted that the liquefaction analysis only pertained to those data points that exhibited a factor of safety of less than 1.1, although according to recommendations in RG 1.198, for an intermediate factor of safety of greater than 1.1 but less than 1.4, stability and deformation analyses should be performed with reduced strength values commensurate with the pore water pressure increase caused by earthquake shaking. Therefore, the staff issued supplemental RAI 02.05.04-28, asking the applicant to discuss: (1) pore-water generation and post-earthquake strength for soils with a factor of safety of less than 1.4; (2) post-earthquake stability of safety-related structures and the potential interaction with adjacent nonsafety-related structures as a result of either liquefaction or strength loss; and (3) factor of safety statistics for the results of each of the three methods used to compute site-wide and structure-specific liquefaction potential. The staff also asked for SPT N-values, CPT tip resistance, and shear wave velocity data in a searchable electronic format in order to perform a confirmatory analysis.

In its response to RAI 02.05.04-28, dated August 10, 2009 (ML092250658), the applicant included an examination of all results from the SPT, CPT, and shear wave velocity measurements. The applicant showed that approximately 98.8 percent of the collected SPT data points indicate that the soils are too strong to liquefy, the factor of safety exceeding 1.1. The applicant also showed that the data points where the factor of safety is less than 1.4 are: (1) located in surficial strata that will be removed during construction, (2) located where no structures will be built, or (3) located under nonsafety-related structures. The applicant identifies a small number of data points with a factor of safety in the intermediate range that exist under safety-related seismic Category I structures. For these post-construction data points, the applicant demonstrated that the points are either at depths great enough not to affect the behavior of the foundation; or the points represent small, localized areas of liquefiable soils that are surrounded by dense, non-liquefiable soils. The applicant also stated that the localized, potentially liquefiable zones are capable of being spanned by the large mat foundations. The applicant's analysis of the data concludes that the low factor of safety points do not illustrate a distinct pattern, congregate, or overlap to a degree that would impair site stability.

To support the conclusion stated above, the applicant provides tables and figures demonstrating that the soils with a factor of safety of less than 1.4 are few and are not congregated in one area. The applicant submited plots of the spatial distribution of factor of safety values of less than 1.1 and between 1.1 and 1.4 for each stratum, and for each of the three methods used to compute the liquefaction potential. Additionally, the applicant provided in tabular form the disposition of all points with factor of safety values of less than 1.4 for each evaluation method-regardless of whether the soil will be excavated or remain in place—and if the soil is left in place, whether it will support a safety-related structure, a nonsafety-related structure, or no structure. SER Figure 2.5S.4-12 (RAI 02.05.04-28 response, dated August 10, 2009, Figure 50, "Spatial Distribution of FOS < 1.40 Remaining after Fuel Load- All Strata"), shows the boring locations for all data points with a factor of safety of less than 1.4 that will remain after excavation and fuel loading.

In its response to the structure instability and/or structure interaction following a SSE, the applicant stated that a post-earthquake instability of safety-related structures and/or a potential interaction with adjacent nonsafety-related structures is not a possibility because the localities with a factor of safety of less than 1.4 are limited in extent, scattered throughout different strata, and surrounded by stronger materials so that the relatively small size of liquefiable zones cannot cause an unstable condition to develop due to their minor influence on the otherwise stable foundation.

For the pore water pressure generation, the applicant estimated the settlements that would result from the volumetric strain due to pore water pressure generation for soils with factor of safety values of less than 1.4. Using the procedure of Ishihara and Yoshimine (1992), the applicant concluded that given the localized nature and considerable depth of the soils with a factor of safety of less than 1.4 below safety-related structure foundations, the compressions will not likely propagate to the foundation level. SER Table 2.5S.4-8 (RAI 02.05.04-28 response Table 4, "Summary of Liquefaction Induced Compressions at Depth Beneath Nuclear Safety Related Structures"; Table 5, "Summary of Liquefaction Induced Compressions at Depth Beneath Non-Nuclear Safety Related Structures Adjacent to Nuclear Safety Related Structures by the CPT Method"; and Table 6, "Summary of Liquefaction Induced Settlements at Depth Beneath Non-Nuclear Safety Related Structures Adjacent to Nuclear Safety Related Structures by the Vs Method") shows the results for the safety- and nonsafety-related structures, including the settlement computed for the turbine building, based on CPT and shear wave velocity measurements.  The applicant added that for compressions below the nonsafety-related turbine building and given the size of the mat, the estimated settlements could be spanned by the mat's foundation.

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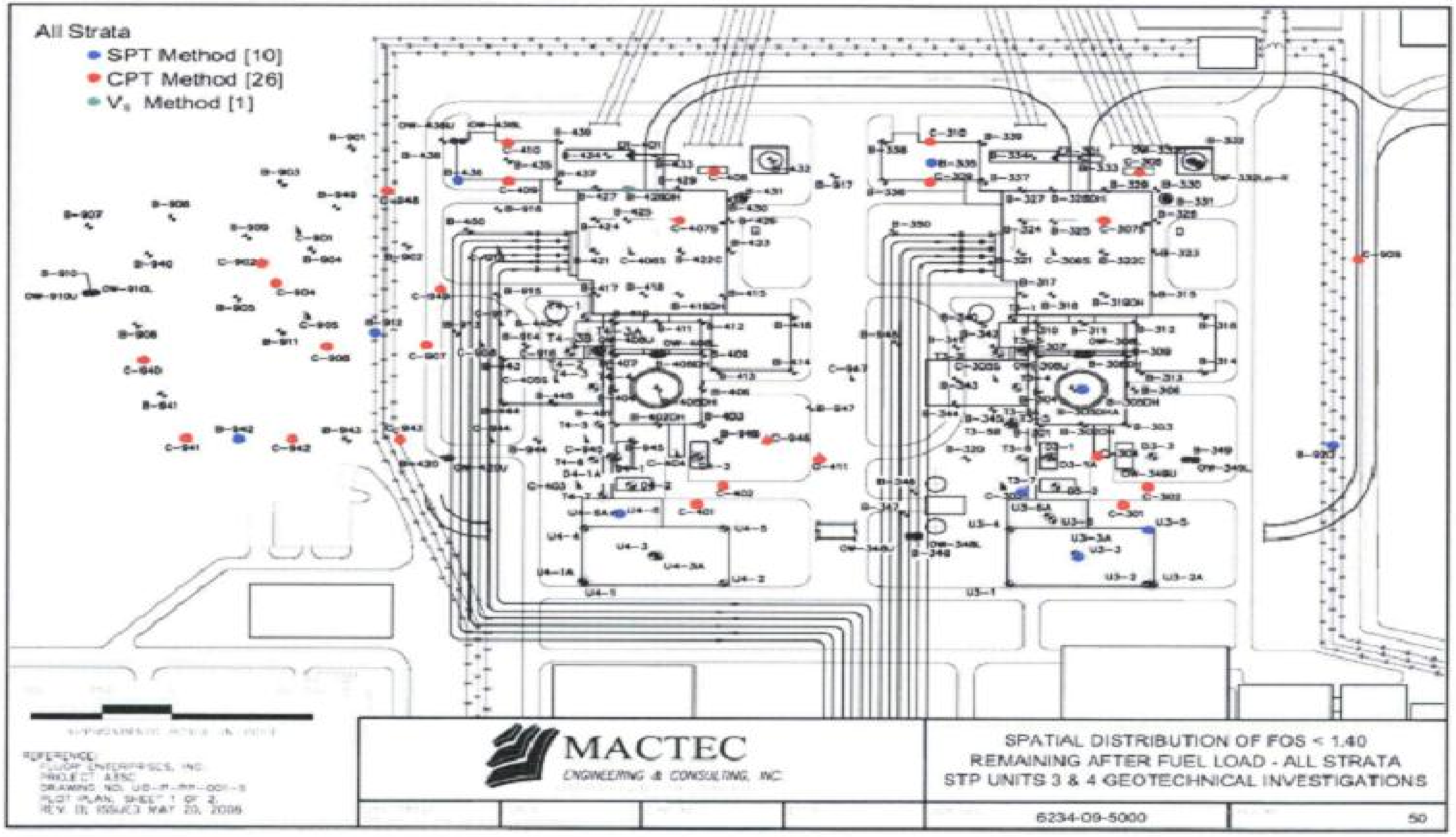


**Figure 2.5S.4-12 Plot of All Data Showing Plan Locations in All Strata of Soils Exhibiting a Factor of Safety Less Than 1.4 for Three Methods of Investigation (RAI 02.05.04-28 Response Figure 50)**

Table 2.5S.4-8 Summary of Liquefaction-Induced Compression Beneath STP Structures
(RAI 02.05.04-28 Response Tables 4, 5, and 6)

| Structure | Foundation El., m (ft) | Boring | Test El., m (ft) | Stratum | Factor of safety | Total Compression, cm (in.) | Depth Below Foundation m (ft) |
|---|---|---|---|---|---|---|---|
| **Safety-Related Structures** | | | | | | | |
| Reactor | -15.24 (-50) | B-305DH | -106.5 (-349.7) | NS2 | 1.16 | 6.096 (2.4) | 91.44 – 97.5 (300 – 320) |
| | | | -112.6 (-369.7) | | 1.37 | | |
| RSW Tunnel | -6.4 (-21) | T3-7 | -58.1 (-190.6) | KSS | 1.04 | 7.1 (2.8) | 51.8 (170) |
| UHS | -1.2 (4) | U3-3 | -11.5 (-38.0) | E | 1.38 | 0.254 (0.1) | 12.8 (42) |
| UHS | -1.2 (4) | U3-5 | -55.9 (-183.6) | KSS | 1.38 | 2.032 (0.8) | 57.3 – 60.3 (188 – 198) |
| | | | -58.9 (-193.5) | | 1.10 | | |
| RSW Pump House | -7.9 (-26) | U4-6 | -72.9 (-239.2) | L | 1.32 | 1.016 (0.4) | 64.3 (211) |
| **Nonsafety-Related Structures** | | | | | | | |
| Turbine Building* | -7.9 (26) | C-307S | -10.2 (-33.7) | D | 1.28 | 0.0508 (0.02) | 2.13 (7) |

| Structure | Foundation El., m (ft) | Boring | Test El., m (ft) | Stratum | Factor of safety | Total Compression, cm (in.) | Depth Below Foundation m (ft) |
|---|---|---|---|---|---|---|---|
| Turbine Building* | -7.9 (-26) | C-307S | -14 (-46.0) | E | 1.32 | 0.254 (0.10) | 6.096 to 8.22 (20 to 27) |
| | | | -14.3 (-47.0) | | 1.33 | | |
| | | | -15.8 (-51.9) | | 1.30 | | |
| | | | -15.9 (-52.4) | | 1.26 | | |
| Turbine Building* | -7.9 (-26) | C-407S | -17.3 (-57) | H | 1.38 | 0.0508 (0.02) | 9.44 (31) |
| Turbine Building# | -2.4 (-8) | B428DH | -4.1 (-13.4) | D | 1.15 | 0.0762 (0.03) | 1.64 (5.4) |

RSW=reactor building service water; UHS=ultimate heat sink; ft=foot; m=meter; cm=centimeter; in.=inch; El=elevation; CPT=cone penetrometer test
# Shear Wave method,
* CPT Method

The applicant also presented histograms illustrating the site-wide factor of safety distribution for each stratum and showing the factor of safety at the location of each safety-related structure encountered in the borings. The applicant noted that an evaluation of the histograms does not reveal a definitive stratum possessing a majority of factor of safety values of less than 1.4.

The applicant concluded that the Beaumont formation is geologically old, and the incorporation of a correction factor for age will likely increase factors of safety above the 1.4 threshold. The applicant further concluded that although quantitative data to support an age correction factor are lacking, the age of the deposit is sometimes accounted for by omitting the correction factor for stress (Ks factor). Using a simple calculation, the applicant demonstrates that omitting the Ks factor is substantial enough to increase computed factors of safety above 1.4. The staff concluded this is acceptable. Therefore, RAI 02.05.04-5 and RAI 02.05.04-28 are resolved and closed

The staff reviewed the applicant's analysis of the data and concluded that the applicant has adequately demonstrated that most of the data points falling below a factor of safety of 1.1 are either non-liquefiable clays, will be excavated during construction, or are located in areas where no structures will be built. The staff also noted that the two data points with a factor of safety of less than 1.1 post-construction are at a significant depth. The staff thus concurred with the applicant that the likelihood of liquefaction at those high overburden pressures is questionable. Additionally, the applicant calculated the settlement that liquefaction would induce. The staff agreed with the applicant's conclusion that because of the depths below the building foundations, liquefaction-induced settlement would not be observed at the foundation level because of the bridging effect of the overlying non-liquefied soils.

The staff also concluded that the applicant has adequately addressed strength degradation due to pore water pressures generated by the SSE, in soils with an intermediate factor of safety of greater than 1.1 and less than 1.4. In addition, the applicant has shown that regarding the distribution of the materials with a factor of safety of less than 1.4, they are not congregated into any one stratum and are surrounded by materials with factor of safety values greater than 1.4. The staff concurred with the applicant that the soils exhibiting factor of safety in the intermediate range are limited in an areal extent and because stronger materials surround these lower factor of safety zones, instabilities are not possible. Furthermore, in reviewing the data, the staff noted that the factor of safety less than 1.4 are typically greater than those of 1.3, thus indicating that very little pore water pressure will be generated in those soils. The staff also noted that although liquefaction is theoretically possible at depths greater than 60.96 m (200 ft) below the surface, it is not very likely due to the high confining pressures. Because it is reasonable to conclude that the points with an intermediate factor of safety represent localized volumes of slightly lower factor of safety soils surrounded by denser materials, the staff concluded that the large mat supporting the Turbine Building should be capable of spanning any localized soft zones induced by the SSE.

As part of reviewing the response to RAI 02.05.04-28, the staff performed a confirmatory analysis to test the accuracy of the applicant's calculations. An independent liquefaction analysis demonstrated the accuracy of the applicant's computations. In general, the staff's calculations showed very similar results when using the deterministic methodology and the SPT and CPT data input. The staff concluded that the applicant had carried out the calculations correctly.

Finally, the staff acknowledges that Pleistocene-age deposits are more resistant to liquefaction than younger soil deposits. The staff calculated that an age adjustment factor of 1.35 would adjust the lowest factor of safety shown in SER Table 2.5S.4-1 to a factor of safety of above 1.4. Because the stress factor sometimes applied to account for age of the deposit ranges from 1.45 to 1.67 for the depths of marginal factor of safety under consideration, the staff concludes that this exceeds the 1.35 needed to raise all factors of safety above the 1.4 threshold. However, since there is no professional consensus on a quantitative correction factor to account for the age of the deposit, the staff concludes that the applicant was conservative in not applying a correction factor to the analysis results. The level of conservatism is uncertain, though its influence is at least qualitatively recognized.

The staff considered: (1) the scattered limited zones of potentially liquefiable soils surrounded by dense non-liquefiable soils, (2) the depths of the liquefiable zones below the foundation levels of the structures, and (3) the scarcity of low factors of safety data points compared to the large number of points collected. The staff concluded that the potential for liquefaction is negligible, and its potential effect on safety-related structures is minor.

Therefore, the staff concluded that for all intents and purposes, liquefaction does not occur at the STP site. Furthermore, the staff's confirmatory analysis validated the accuracy of the applicant's results. Therefore, RAI 02.05.04-28 and RAI 02.05.04-5, are resolved and closed.

FSAR Subsection 2.5S.4.8, also addresses COL License Information Item 2.33, requiring the COL applicant referencing the ABWR DCD to verify that there is no potential for liquefaction in the soils underlying the seismic Category I structures at the STP site. The applicant performed a liquefaction analysis and concluded that although there are some data points that exhibit a potential for liquefying, these points are not sufficient in number or in proximity to one another to pose a threat to the seismic Category I structures. Furthermore, the staff performed an independent confirmatory analysis that validated the accuracy of the applicant's results—liquefaction does not occur at the STP site. Because the applicant has demonstrated and the staff has confirmed that liquefaction does not occur at this site, the staff concluded that the applicant's information sufficiently addresses COL License Information Item 2.33.

The staff reviewed FSAR Subsection 2.5S.4.8, and concluded that the applicant's liquefaction analysis is complete and accurate and is supported by the staff's independent confirmatory analysis. In addition to the results of the confirmatory analysis, the staff concluded that the use of CPT, SPT and shear wave velocity data as part of the liquefaction analysis forms an adequate basis for COL License Information Item 2.33. The staff also concluded that the applicant's liquefaction analysis in FSAR Subsection 2.5.S.4.8, forms an adequate basis for assessing the liquefaction potential at the STP site and meets the regulatory requirements of 10 CFR Parts 50 and 100.

2.5S.4.4.9 Earthquake Design Basis

FSAR Subsection 2.5.4.9, "Earthquake Design Basis," refers to Subsection 2.5S.2.6 for a detailed discussion of the GMRS. SER Subsection 2.5S.2.4.6, includes a detailed evaluation of FSAR Subsection 2.5S.4.9.

2.5S.4.4.10 Static Stability

In FSAR Subsection 2.5.4.10, the applicant describes the foundation design of the STP, Units 3 and 4, seismic Category I structures including bearing capacity, settlement, and lateral earth pressures on buried walls.

**Bearing Capacity**

The staff reviewed the applicant's bearing capacity assumptions, methods, and results. The staff concurred with the selected material properties for analysis and the weighted averaging technique used to simplify the multilayered system into a more convenient form for the bearing capacity analysis. The staff concluded that it is appropriate to perform stability calculations for two loading cases, end of construction, and long-term loading conditions where seismic forces are considered. The staff also concluded that the water table assumptions for each case are adequate. Furthermore, the staff found that the applicant was conservative in developing specific subsurface profiles for each major structure using the most susceptible soil stratum beneath the foundation rather than the average layering conditions. However, to complete a review of the applicant's information, the staff noted the need for a discussion of the assumptions and soil properties used to compute the factor of safety for seismic Category I structures against the dynamic bearing capacity and sample calculations of the dynamic bearing capacity determination. Therefore, the staff issued RAI 02.05.04-15, asking the applicant to discuss the assumptions and soil properties used to compute the factor of safety for seismic Category I structures against the dynamic bearing capacity, and to provide a sample calculation of the dynamic bearing capacity determination.

In its response to RAI 02.05.04-15, dated January 28, 2009 (ML090300648), the applicant provided a sample calculation showing the methodology used to determine the dynamic bearing capacity. However, the applicant's response does not report the calculated factor of safety under SSE dynamic loading conditions for all seismic Category I structures. The staff reviewed the applicant's method and concluded that the quasi-static method is conservative, because it takes a transient load and applies it statically to a reduced foundation footprint that accounts for the eccentric loading produced by the SSE. The applicant also cited a criterion factor of safety of 1.5 when dynamic or transient loading conditions apply. The staff noted that the cited factor of safety of 1.5 is lower than the factor of safety for the transient loading of 2.0, which is commonly referenced in standard geotechnical textbooks. Therefore, the staff issued supplemental RAI 02.05.04-29, requesting the applicant to provide the factor of safety for the safety-related structures at STP, Units 3 and 4, under the dynamic SSE loading, and to justify the use of a factor of safety of 1.5 for dynamic loading.

In its response to RAI 02.05.04-29, dated September 21, 2009 (ML092710096), the applicant stated that the site-specific seismic analysis of the Reactor and Control Buildings and the UHS/RSW Pump Houses is currently under investigation and the factors of safety for these safety-related structures for the dynamic bearing capacity for the site-specific conditions will be submitted at a later date as part of a supplemental RAI response.

The applicant justified the use of a factor of safety of 1.5 because the ABWR DCD does not specify any requirements for an acceptable dynamic bearing capacity factor of safety. The applicant derived the factor of safety from ASCE (1980), which is applicable to nuclear power plants. The applicant also noted that RG 1.198 specifies 1.4 as an acceptable factor of safety for soil liquefaction, which the applicant concluded is acceptable because soil bearing and soil liquefaction are similar in importance with respect to foundation stability. Furthermore, the applicant stated that the SSE is a very short duration load with the peak loading acting momentarily and decreasing rapidly, thus permitting only limited soil deformations even with a factor of safety approaching 1.0 or lower. Finally, the applicant stated that the peak dynamic bearing pressure for the SSE loading is at a corner of the foundation, with the dynamic bearing pressure decreasing rapidly away from the foundation corner and making average loading significantly smaller than the peak corner loading. For these reasons, the applicant concluded that a factor of safety of 1.5 under those conditions is acceptable.

The staff reviewed this information and agrees that the small area over which the peak loading occurs cannot result in a generalized bearing capacity failure, and the liquefaction factor of safety of 1.4 is a reliable minimum factor of safety for comparison because it suggests a level of stability at which deformations resulting from dynamic loading will be negligible. Given the relatively low seismic demand, the staff concluded that the factor of safety of 1.5 is a sufficient level of safety for this dynamic bearing capacity loading case based on the transient and localized dynamic loading conditions. Therefore, RAI 02.05.04-15 is resolved and closed. However, the staff considered RAI 02.05.04-29, unresolved until the applicant satisfactorily completed the dynamic bearing capacity analyses for all safety-related structures and provides the factor of safety for the safety-related structures.

The staff issued supplemental RAI 02.05.04-35, requesting the applicant to provide the dynamic bearing capacity factor of safety for all seismic Category I structures, or justify why sufficient margin exists for some structures such that performing these analyses is not necessary.

In its response to RAI 02.05.04-35, dated April 27, 2010 (ML101270284), the applicant revised Table 2.5S.4-41C, “Bearing Capacity of Foundations under Dynamic or Transient Loading,” to include the dynamic bearing capacity factors of safety for the RSW piping tunnels and the diesel generator fuel oil storage vaults. Although the applicant does not provide the factor of safety for the diesel generator fuel oil tunnels, when compared to the factors of safety for the RSW piping tunnels and diesel generator fuel oil storage vaults, which are in excess of 30, the applicant concluded that these structures are lightly loaded and the factor of safety would be greater than the required value of 1.5.

The staff has reviewed the applicant’s response and concludes that factors of safety were reported for all of the seismic Category I structures. The staff noted that the static factors of safety were all typically greater than 3.0 and the dynamic factors of safety were generally 2.0 or greater, the only exception being the factor of safety for dynamic bearing capacity for the STP, Unit 4, Control Building, which was given as 1.73. Because the given factor of safety exceeds the acceptable factor of safety given in ASCE (1980) as referenced in the FSAR, the staff concludes that a factor of safety of 1.73 is adequate. The staff confirmed that the applicant revised Table 2.5S.4-41C, to include the dynamic bearing capacity factors of safety for the RSW piping tunnels and the diesel generator fuel oil storage vaults and this information was included in COL FSAR Revision 4. Therefore, RAI 02.05.04-29 and RAI 02.05.04-35 are resolved and closed.

The applicant’s results show that the static factor of safety values are typically greater than 3.0 for all seismic Category I structures, which the staff noted is a commonly accepted factor of safety for important structures throughout the industry. The staff performed a confirmatory analysis using the U.S. Army Corps of Engineers’ bearing capacity computer program CBEAR. The staff’s analysis obtained a factor of safety of 3.1 for the end of construction case for STP, Unit 3, which compares favorably with the applicant's factor of safety of 3.03 for the same structure and case loading. Based on the staff's review of the applicant's material properties, analytical methods, and factor of safety values, as well as the results of the confirmatory analysis, the staff concluded that the static bearing capacity of the seismic Category I structures for STP, Units 3 and 4, meets or exceeds the design requirements of the ABWR DCD.

**Settlement**

The applicant estimated the settlement of the seismic Category I structures using the material properties developed in FSAR Subsection 2.5S.4.2. The applicant found that the settlement was primarily pseudo-elastic due to the overconsolidated state of the soil strata, with only minor consolidation settlement occurring under specific structures and loading conditions. FSAR Table 2.5S.4-42, "Estimated Foundation Settlements," presents the settlement predictions for total and differential settlement and tilt.

In FSAR Subsection 2.5S.4.10.4, the applicant described the potential settlement for STP, Units 3 and 4. However, it was not clear to the staff that overlapping stresses from adjacent buildings were considered in the calculations. The staff issued RAI 02.05.04-13, requesting the applicant to discuss the underlying assumptions of the estimated settlement and heave and to provide a sample calculation of settlement and heave under STP, Units 3 and 4.

In its response to RAI 02.05.04-13, dated January 28, 2009 (ML091820695), the applicant clarified that the estimated foundation settlements are premised on pseudo-elastic compression and one-dimensional consolidation for all the seismic Category I structures in the STP, Units 3 and 4, power block areas. The applicant's assumptions included a Boussinesq-type stress distribution below rectangular, flexible foundations extending to a depth of 762 m (2,500 ft) to capture overlapping stresses from all contributing structures. FSAR Table 2.5S.4-42, shows these settlement estimates.

The applicant's calculations for total settlements at the centers of foundations are 25.6 to 27.1 cm (10.1 to 10.7 in.) for the Reactor Buildings; 19.8 to 21.1 cm (7.8 to 8.3 in.) for the Control Buildings; 20.8 to 21.6 cm (8.2 to 8.5 in.) for the UHS Basins; 17.8 to 18.3 cm (7.0 to 7.2 in.) for the RSW Pump Houses; 29.9 to 30.48 cm (11.8 to 12.0 in.) for the RSW Tunnels; and 14.7 to 20.1 cm (5.8 to 7.9 in.) for the Diesel Generator Fuel Oil Storage Vaults. The applicant noted that some of the settlements are overstated because these values assume no buoyancy on the structures. The applicant also predicted that the soil heave resulting from the 27.4 to 28.9 m (90 to 95 ft) of excavation at the Reactor Buildings would be in the range of approximately 8.9 to 16.5 cm (3.5 to 6.5 in.). SER Table 2.5S.4-9 (FSAR Section 2.5S.4.10.4) shows the estimated differential settlement and the angular distortion/tilt values.

**Table 2.5S.4-9 Estimated Differential Settlement and Distortion/Tilt (FSAR Subsection 2.5S.4.10.4)**

| Structure | Flexible Differential Settlement cm (in.) | Estimated Maximum Flexible Angular Distortion/Tilt |
|---|---|---|
| Reactor Buildings | 3.8 to 4.5 (1.5 to 1.8) | 1/600 to 1/750 |
| Control Building | 4.5 to 5.08 (1.8 to 2.0) | 1/400 to 1/450 |

| Structure | Flexible Differential Settlement cm (in.) | Estimated Maximum Flexible Angular Distortion/Tilt |
|---|---|---|
| UHS Basins | 5.6 to 5.8 (2.2 to 2.3) | 1/650 to 1/700 |
| RSW Pump Houses | 1.27 (0.5) | 1/1700 to 1/1750 |
| RSW Tunnels | 12.7 (5.0) | 1/700 |
| Diesel Generator Fuel Oil Storage Vaults (No. 1) | 1.27 (0.5) | 1/1000 to 1/1050 |
| Diesel Generator Fuel Oil Storage Vaults (No. 2) | 1.27 (0.5) | 1/500 to 1/550 |
| Diesel Generator Fuel Oil Storage Vaults (No. 3) | 1.016 (0.4) | 1/650 to 1/750 |
| RSW=reactor building service water, UHS=ultimate heat sink | | |

The applicant plans to mitigate the differential settlement by superstructure and mat rigidity. The applicant estimated that the differential settlement of a rigid foundation may be one-half or less than that calculated for a flexible foundation. In addition, the applicant expected the actual angular distortion/tilt values to be much less given that one-half or more of the foundation settlements are expected to take place by the time the building superstructures are ready to receive equipment and/or piping. To that end, the applicant recalculated the estimated angular distortion/tilt to be one-half of the earlier calculations. The applicant found that these distortion/tilt values are well within the criterion of 1/750 for foundations supporting sensitive machinery. The applicant plans to develop acceptance criteria for the settlement of seismic Category I structures during the detailed design stage and to monitor major structure foundations for movement during and after construction. The applicant also described plans to evaluate the effects of construction sequencing on the time-rate of settlement using the settlement monitoring program. The applicant will adjust the scheduling to minimize adverse effects on the structural and mechanical SSCs.

The staff reviewed the material property assumptions, analytical methods, and results and using computer program Settle 3D 2.0, performed a confirmatory settlement analysis for the center point under the STP, Unit 3, Reactor Building to check the accuracy of the spreadsheet calculations. The staff concluded that the applicant’s analytical procedures are correct. The staff concurred with the applicant that differential settlement and distortion/tilt is generally more critical than the total settlement of an individual structure, and some portion of the settlement will occur before setting equipment or making piping connections. Because the applicant will monitor the settlements, the staff concluded that the applicant will be able to observe when the settlements are leveling out and will wait for the appropriate time to proceed with utility connections between structures. The staff also concurred that the settlement predictions based on flexible basemats will overpredict actual settlements of a rigid foundation, and the differential settlement of individual structures could be one-half or less of the predicted settlements. Although the actual differential settlements will have to be confirmed by monitoring the settlement, the staff concurred that distortion will be less than predicted, and because equipment mounting will occur late in the schedule, most settlement should occur and any distortion or tilt should be accommodated as a matter of construction or by field modifications. Finally, the staff concluded that careful monitoring, construction sequencing, and minor field modifications will accommodate the actual total and differential settlements. Therefore, RAI 02.05.04-13 is resolved and closed.

In a letter dated December 20, 2007 (ML073580003), the applicant stated the intent to develop a program that will manage settlement and differential settlement; the applicant committed to share this program with the NRC. The staff issued RAI 02.05.04-21, asking the applicant to describe the acceptance criteria and methods used to ensure that all settlement is complete before fuel loading. The staff also asked the applicant to describe how to ensure that no excessive stresses will result from the settlements and differential settlements within and/or between safety-related structures, in any SSCs of the seismic Category I structures.

In its response to RAI 02.05.04-21, dated April 1, 2009 (ML090930717), the applicant added language that refers to construction sequencing and acceptance criteria for settlement, but does not provide sufficient detail for the staff to complete a review. Accordingly, the staff issued supplemental RAI 02.05.04-30, asking the applicant to: (1) elaborate on the means of using construction sequencing to evaluate the time-rate of settlement; (2) provide the settlement criteria for fuel loading, including a discussion of how to ensure that the settlement after fuel loading will not be damaging settlements; and (3) define the specific DCD acceptance criteria to be followed.

In its response to RAI 02.05.04-30, dated September 21, 2009 (ML092710096), the applicant stated that the criteria for differential settlement and tilt values used for analysis and design are based on the post-construction settlement values. The applicant will use settlement predictions as a baseline for comparison with actual settlements and plans to update settlement predictions using real-time construction and geotechnical data to ensure that post-construction settlement predictions used to design the SSCs remain within the criteria established. The applicant also plans to evaluate any variation in actual settlement versus the predicted settlement and will adjust the schedule or construction sequencing to mitigate damage. The applicant also stated that in order to protect the safety-related SSCs from potentially damaging settlements, these settlements occurring after fuel loading will be documented in an engineering study that will predict the magnitude of future settlements and show that the predicted settlements are within the design values. The staff concurred with the applicant’s plan to establish a baseline for the time-rate of settlement through calculation and to periodically update the calculation with real-time construction data compiled from monitoring settlements during construction.

The staff concluded that the method used to modify construction plans to mitigate settlements and ensure that actual post-construction settlement will be within the tolerances used in the design of the safety-related SSCs is adequate. Furthermore, the staff reviewed the applicant’s ITAAC and concluded that they are sufficient to evaluate the settlement of the seismic Category I structures and associated systems and components, thus ensuring that post-construction settlement after fuel loading will not adversely impact the SSCs.

In its response to RAI 02.05.04-34, dated April 27, 2010 (ML101270284), and RAI 02.05.04-36, dated June 3, 2010 (101590397), the applicant revised the settlement ITAAC provided in response to RAI 02.05.04-30, to provide greater specificity regarding the tests and quantitative acceptance criteria. SER Table 2.5S.4-10, provides the ITAAC for settlement prior to fuel load proposed by the applicant, and replaces the previously-proposed settlement ITAAC that applies after fuel load.

**Table 2.5S.4-10 ITAAC for Settlement before Fuel Load (COL Application Part 9, Table 3.0-15)**

| Design commitment | Inspections, test, and analyses | Acceptance criteria |
|---|---|---|
| 1. Settlement of structures measured three (3) months prior to fuel load shall be less than the values in the acceptance criterion. | 1. Field measurements of actual settlement of seismic Category I structures will be taken three (3) months prior to fuel load. | 1. Maximum allowable tilt (defined as the differential settlement between two edges on the centerline axes of a structure divided by the lateral dimension between these two points) is 1/600. |

The staff noted that the settlement ITAAC now includes the settlement criteria that the applicant used to design the mat foundations such that the actual settlement of the basemat can now be compared to the design value, and greater tilt than 1/600 would require evaluation of the basemat performance. The staff therefore concludes that the settlement ITAAC is acceptable to verify that the actual settlement does not exceed the values assumed in the design analysis. Based on the information the applicant has provided, including the settlement ITAAC, the staff found that this approach is acceptable and will ensure the post-construction safety and stability the safety-related SSCs. Therefore, RAI 02.05.04-21 RAI 02.05.04-30, RAI 02.05.04-34 and RAI 02.05.04-36, are resolved and closed.

**Lateral Earth Pressures**

The staff reviewed FSAR Subsection 2.5.4.10.5, including the static and dynamic lateral earth pressures and the sample earth pressure diagrams from FSAR Figures 2.5S.4-76, “Sample Active Lateral Earth Pressure Diagrams,” and 2.5S.4-77, “Sample At-Rest Lateral Earth Pressure Diagrams,” for the maximum 25.9-m (85-ft) wall height assuming level ground surface conditions behind the wall, and a ground water level at the ground surface. In order to compute the lateral earth pressures, the applicant assumed soil properties for the backfill materials because the source of the backfill has not been determined. The applicant also committed to include the final earth pressure calculations, following completion of the project detailed design, in an update to the FSAR in accordance with 10 CFR 50.71(e) (COM 2.5S-3). The staff reviewed the applicant’s assumptions for ground water location; the estimated earth pressure coefficients based on Jaky’s relationship (Jaky, 1948); the assumed friction angle; and the analytical methods. The staff concluded that these assumptions are all conservative.

FSAR Subsection 2.5S.4.10.5.2, describes the determination of the seismic lateral earth pressures. The staff issued RAI 02.05.04-14, requesting the applicant to provide a sample calculation of the dynamic lateral stress computation.

In its response to RAI 02.05.04-14, dated January 28, 2009 (ML090300648), the applicant provided sample calculations using the Ostadan (2004) procedure and the SHAKE computer program to calculate dynamic lateral stresses against deeply embedded below ground walls of heavy structures, such as the reactor building and the control building. The applicant used the Elastic Solution, which is described in Subsection 3.5.3.2.2 of ASCE 4-98, to calculate lateral stresses against shallow embedded lightweight structures such as the RSW Tunnels, the UHS basin, and the RSW pump house. The applicant selected the Ostadan method and ASCE 4-98 over the Mononobe-Okabe equation because the former methods are applicable for at-rest earth conditions, whereas the latter applies to the case where walls are free to deflect, thus resulting in less conservative results.

The applicant used the computer program SHAKE to determine the acceleration of the soil column at the base of the wall. With the acceleration at the base of the wall determined, and the total mass for a representative backfill soil column computed, the applicant calculated the total lateral seismic force on the wall by multiplying the soil mass by the spectral acceleration at the natural frequency of the backfill. Finally, the applicant computed the lateral seismic soil pressure distribution along the height of the wall.

The staff reviewed the calculations and concluded that the use of the Ostadan and ASCE 4-98 methods is appropriate and more conservative than the Mononobe-Okabe method because the selected methods consider the rigidity and weight of the structure, the embedment depth, and the frequency content of the strong ground motion, thereby resulting in adequately conservative results. Therefore, RAI 02.05.04-14 is resolved and closed.

*COL License Information Items 2.35, 2.36, 2.37, 2.38, and 2.39*

FSAR Subsection 2.5S.4.10, also addresses COL License Information Items 2.35, 2.36, 2.37, 2.38, and 2.39, requiring the COL applicant referencing the ABWR DCD to: (1) verify that the site meets the minimum static bearing capacity of 718 kPa (15 ksf); (2) evaluate the lateral earth pressures, (3) justify the soil properties used for the seismic analysis of buried pipes and conduits; (4) perform a stability evaluation; and (5) describe the subsurface instrumentation used to monitor the foundations of safety-related structures. The applicant has confirmed the static and dynamic bearing capacity and stability of the structures. RAI 02.05.04-29, referred to the dynamic bearing capacity of all Seismic Category 1 structures and was resolved by the response to RAI 02.05.04-35. The applicant also evaluated the earth pressures. As part of the settlement monitoring program, the applicant will delay the installation of pipes and conduits between buildings until the majority of the settlement has occurred. Finally, the applicant described the monitoring program, including the subsurface instrumentation in detail. The staff concluded that the applicant's information adequately addresses COL License Information Items 2.35, 2.36, 2.37, 2.38, and 2.39.

**Staff Conclusions Regarding Static Stability**

The staff reviewed FSAR Subsection 2.5S.4.10, and concluded that the applicant has developed an accurate assessment of the static stability at the STP site that addresses COL License Information Items 2.35, 2.36, 2.37, 2.38, and 2.39, including the minimum static bearing capacity; earth pressures; seismic analysis of buried pipes; static stability of facilities; and subsurface instrumentation. The staff concluded that the dynamic bearing capacity calculations provided by the applicant adequately resolve RAIs 02.05.04-29 and 02.05.04-35, and address COL License Information Item 2.38. Accordingly, the staff concludes that the applicant's information in FSAR Subsection 2.5S.4.10—the bearing capacity determination, lateral earth pressure calculations, and settlement estimations—forms an adequate basis for the static stability at the site and meets the requirements of 10 CFR Parts 50 and 100.

2.5S.4.4.11 Design Criteria

The staff reviewed FSAR Subsection 2.5S.4.11, and concluded that because the applicant has provided the factor of safety for the dynamic bearing capacity sufficient to resolve RAI 02.05.04-29, the applicant has provided adequate factors of safety and design criteria to ensure the safety of the SSCs at the STP site area. The staff also concluded that the applicant's design values described in FSAR Subsection 2.5S.4.11, form an adequate basis for the design criteria and meet the design values of the ABWR DCD and the requirements of 10 CFR Part 50.

2.5S.4.4.12 Techniques to Improve Subsurface Conditions

The applicant limits the ground treatment to localized overexcavation of unsuitable soils at foundation subgrades and their replacement with concrete backfill. The applicant plans a 3.05 m (10 ft) overexcavation of Stratum F at the STP, Units 3 and 4, Reactor Buildings and a general overexcavation of 0.61 m (2 ft) at the control buildings, UHS basins, RSW tunnels, RSW pump houses, and diesel generator fuel oil storage vaults. After subgrade preparation the applicant plans to backfill the overexcavated areas with concrete.

The staff reviewed FSAR Subsection 2.5S.4.12, and concluded that the applicant has described the complete plans for improving and monitoring the subsurface conditions at the STP site. The staff also concluded that the applicant's methods of improvement and monitoring plans described in FSAR Subsection 2.5S.4.12, form an adequate basis for improving subsurface conditions at the site and meet the requirements of 10 CFR Part 50.

### *2.5S.4.5 Post Combined License Activities*

The applicant identifies the following commitment:

- Commitment (COM 2.5S-3) –Update the FSAR in accordance with 10 CFR 50.71(e) to provide the final earth pressure calculations following completion of the project detailed design.
- The applicant also provides the Settlement ITAAC as indicated in SER Table 2.5S.4-10. Furthermore, the applicant provides a three-part Backfill ITAAC as indicated in the SER Table 2.5S.4-8, which addresses backfill properties, shear wave velocity and engineering properties of backfill.

### *2.5S.4.6 Conclusion*

The staff reviewed the application, and confirmed that the applicant has addressed the required information relating to the stability of subsurface materials and foundations, and no outstanding information is expected to be addressed in the COL FSAR related to this section.

The staff reviewed the application and concluded that the applicant has met the regulatory requirements of 10 CFR Parts 50 and 100 and has followed the guidance of RG 1.132, RG 1.138, RG 1.198, and RG 1.206. The applicant has performed an adequate subsurface exploration that meets or exceeds the requirements for numbers and depths of borings. The applicant uses various field exploratory methods to confirm soil properties between methods. The applicant has also performed laboratory tests in satisfactory numbers and types of tests to adequately characterize the static and dynamic properties of in situ site soils. The applicant satisfactorily documents field and laboratory test procedures. The staff finds that the soil properties used in the analyses represent the actual site conditions beneath the planned locations of the plant facilities based on the soil data in FSAR Subsection 2.5S.4.2. The staff finds that the methods of analysis are appropriate for the planned foundations and soil conditions at the site. The methods of analysis for determining bearing capacity as well as settlement, and static and dynamic lateral loads were reviewed for agreement with the state-of-the-art methods, the use of appropriate factors of safety, and consistency with the assumptions made in the development of the methods of analysis.

Static analyses of the bearing capacity and the settlement of the supporting soils under the loads of fill and foundations were evaluated using conventional, state-of-the-art methods. In general, the staff confirmed that the applicant's evaluation procedures were conservative and included conventional factors of safety.

The staff finds that the applicant has carefully considered the design criteria and has incorporated adequate measures in Quality Assurance Programs to ensure tolerable post-construction settlements. The staff's own settlement, bearing capacity, and liquefaction confirmatory analyses matched portions of the applicant's analyses. The applicant has completed the dynamic bearing capacity analyses providing factors of safety for all seismic Category I structures. The staff reviewed the applicant's sample calculation for determining dynamic bearing capacity and agrees with the procedure. Therefore, the staff finds that the computed factors of safety are adequate. The applicant has not identified the backfill source(s) or all of the information relevant to ensure the proper placement of the backfill. The performance of field and laboratory tests to determine that the static and dynamic soil properties are bounded by the assumptions made by the applicant in the design and analysis of the foundations are still required.

The staff finds that the applicant has provided adequate information to address the COL License Information Items pertaining to FSAR Section 2.5S.4, particularly COL License Information Item 2.26, requiring information to address the properties and stability of the subsurface materials. The staff finds that the stability of subsurface materials and foundations at the COL site are in accordance with the requirements of 10 CFR 100.23.

## 2.5S.5 Stability of Slopes

### *2.5S.5.1 Introduction*

This section of the FSAR addresses the stability of all earth and rock slopes both natural and manmade (cuts, fill, embankments, dams, etc.) whose failure, under any conditions to which they could be exposed during the life of the plant, could adversely affect the safety of the plant. The staff evaluated the following topics based on data provided by the applicant in the STP COL application and information available from other sources:

(1) Slope characteristics.

(2) Design criteria and design analysis.

(3) Results of the investigations including borings, shafts, pits, trenches, and laboratory tests.

(4) Properties of borrow material, compaction and excavation specifications.

(5) Any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to CFR Part 52.

### *2.5S.5.2 Summary of Application*

In Section 2.5S.5, of the STP, Units 3 and 4, COL FSAR Revision 12, the applicant provides site-specific supplemental information to address COL License Information Items 2.40 and 2.41 identified in DCD Tier 2, Revision 4, Section 2.3.

<u>*COL License Information Items*</u>

- COL License Information Item 2.40 Stability of Slopes

The ABWR DCD states that the COL applicant will provide “information about the static and dynamic stability of all soil and rock slopes, the failure of which could adversely affect the safety of the plant.” In FSAR Section 2.3, “COL License Information,” the applicant stated that the required information is in Section 2.5S.5, “Stability of Slopes.”

- COL License Information Item 2.41 Embankments and Dams

The ABWR DCD states that the “COL applicant should provide information about the static and dynamic stability of all embankments and dams that impound water required for safe operation (and shutdown) of the ABWR for review by the NRC if embankments and dams are used.” The applicant stated that there “are no embankments or dams that impound water required for safe operation (and shutdown).”

The applicant developed FSAR Section 2.5S.5, “Stability of Slopes,” to evaluate slope stability at the STP site based on information derived from site investigations, geotechnical characterization studies, and excavation and backfill profiles in FSAR Subsections 2.5S.4.1 through 2.5S.4.5. The focus of these investigations and studies include geologic features and characteristics; site exploration involving soil and rock boring and sampling, groundwater monitoring, surface geophysical testing, in situ testing, geotechnical test pits, geologic trench excavations, and laboratory testing; and geophysical surveys.

#### 2.5S.5.2.1 Slope Characteristics

In FSAR Subsection 2.5S.5.1, the applicant describes the characteristics of existing permanent slopes. The applicant stated that the site is relatively flat and the only permanent slopes consist of the main cooling reservoir embankment slopes, which were constructed as part of the original STP, Units 1 and 2. The main cooling reservoir is located approximately 610 m (2,000 ft) south of STP, Units 3 and 4, and consists of approximately 65,500 feet of embankment. SER Figure 2.5S.5-1, shows a site plan view that includes the location of the main cooling reservoir embankment. The top of the embankment varies, ranging from an elevation of 20 m to 20.4 m (65.75 ft to 67 ft), with a normal operating reservoir water level at an elevation of 14.9 m (49 ft). The natural ground surface ranges from an elevation of 8.2 m to 8.8 m (27 ft to 29 ft). The applicant stated that the interior embankment slopes are 2.5H:1V, while the exterior slopes are 3H:1V.

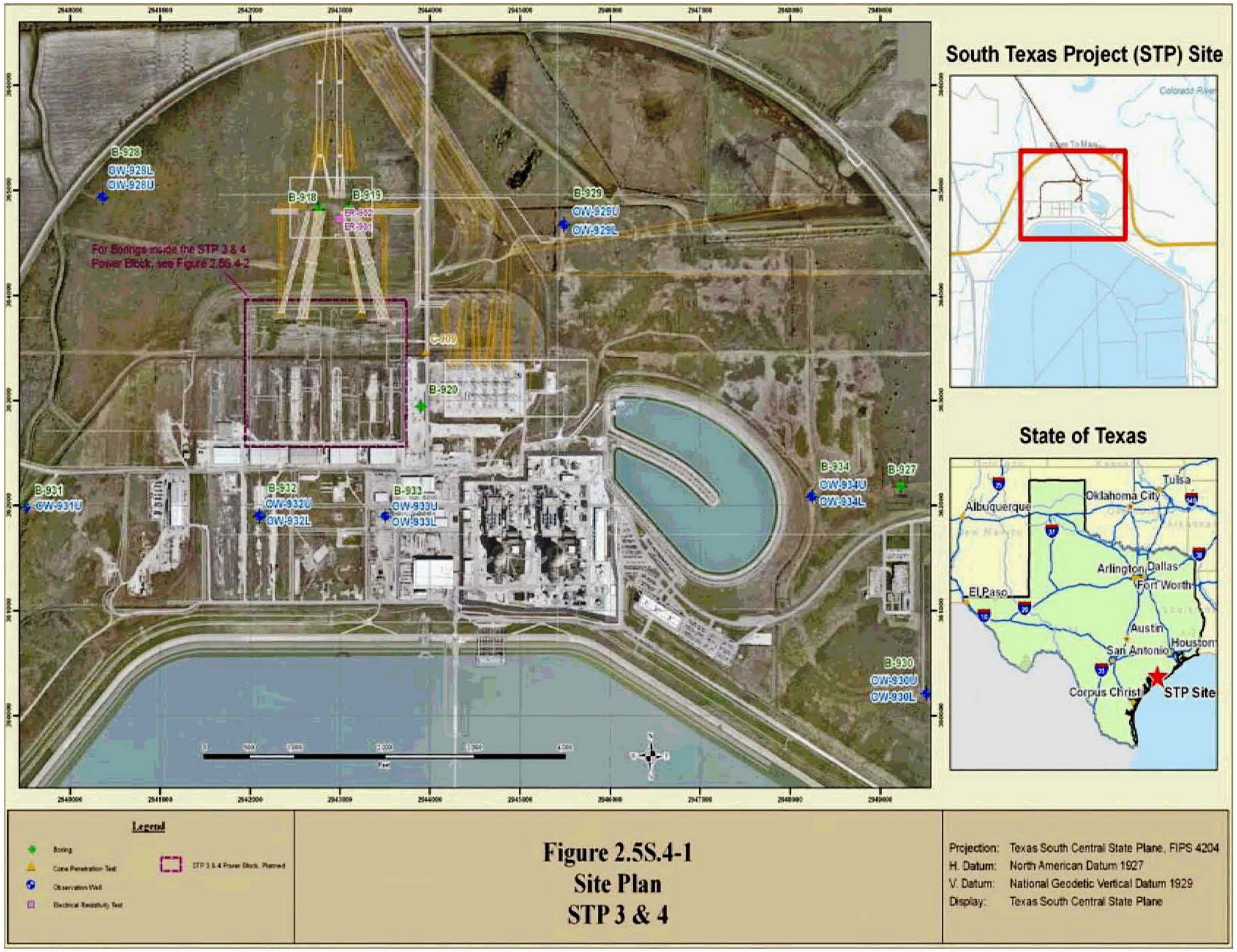


**Figure 2.5S.5-1 Site Plan including Location of the Main Cooling Reservoir Embankment (FSAR Figure 2.5S.4-1)**

2.5S.5.2.2 Design Criteria and Analysis

In FSAR Subsection 2.5S.5.2, the applicant summarized the stability analysis performed for the main cooling reservoir embankment. The complete description of this analysis is in the STP, Units 1 and 2, UFSAR. The applicant noted that the slope stability analysis consists of evaluating the main cooling reservoir embankment for various design conditions and calculating the factors of safety for each case. SER Table 2.5S.5-1, presents the cases evaluated and the calculated factors of safety.

The applicant also considered the potential failure of a 609-meter-long (2,000-foot) embankment section in the flood analysis for STP, Units 3 and 4. The applicant stated that a failure of the main cooling reservoir embankment will not impact the safety of the STP, Units 3 and 4, seismic Category I structures.

**Table 2.5S.5-1 Slope Stability Analysis Considerations**

| Case | Factor Of Safety |
|---|---|
| 1. Reservoir water level set at maximum operating level | 1.7 to 1.8 (exterior slopes)<br>1.8 to 1.9 (interior slopes) |
| 2. Reservoir rapid drawdown analysis | 1.4 to 1.5 |
| 3. Pseudo-static dynamic slope stability analysis | 1.3 to 1.5 |
| 4. Liquefaction potential analysis | 1.1 to 1.6 (OBE)<br>1.7 to 4.7 (SSE) |
| OBE=operating-basis earthquake, SSE=safe-shutdown earthquake | |

2.5S.5.2.3 Boring Logs

In FSAR Subsection 2.5S.5.3, the applicant referred to the STP, Units 1 and 2, UFSAR for the logs of borings and associated references for the main cooling reservoir embankment. Additionally, the applicant provided the logs of borings and information related to field testing corresponding to STP, Units 3 and 4, subsurface investigations in a MACTEC report (2007), “Results of Subsurface Investigation and Laboratory Testing, South Texas Project Units 3 and 4.”

2.5S.5.2.4 Compacted Fill

The applicant stated that FSAR Subsection 2.5S.4.5 addresses the compacted fill requirements.

### *2.5S.5.3 Regulatory Basis*

The relevant requirements of the Commission regulations for the stability slopes, and the associated acceptance criteria, are in Section 2.5.5 of NUREG–0800.

In particular, the applicable regulatory requirements for reviewing the applicant’s discussion of the stability of slopes are the following:

- 10 CFR 50.55a, “Codes and Standards,” requires that SSCs shall be designed, fabricated, erected, constructed, tested, and inspected in accordance with the requirements of the applicable codes and standards commensurate with the importance of the safety function to be performed.

- 10 CFR Part 50, Appendix A, GDC 1, requires that SSCs important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. This criterion also requires that appropriate records of the design, fabrication, erection, and testing of SSCs important to safety be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit.

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

- 10 CFR Part 50, Appendix A, GDC 44 requires that a system be provided with the safety function of transferring the combined heat load from SSCs important to safety to an ultimate heat sink under normal operating and accident conditions.

- 10 CFR Part 50, Appendix B establishes quality assurance requirements for the design, construction, and operation of those SSCs of nuclear power plants that prevent or mitigate the consequences of postulated accidents that could cause undue risks to the health and safety of the public.

- 10 CFR Part 50, Appendix S applies to the design of nuclear power plant SSCs important to safety to withstand the effects of earthquakes.

- 10 CFR Part 100, provides the criteria that guide the evaluation of the suitability of proposed sites for nuclear power and testing reactors.

- 10 CFR 100.23, provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and to identify geologic and seismic factors that must be taken into account when siting and designing nuclear power plants.

The related acceptance criteria are summarized in SRP Section 2.5.5:

- <u>Slope Characteristics</u>: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of slope characteristics is acceptable if the subsection includes the following:

  a. Cross sections and profiles of the slope in sufficient quantity and detail to represent the slope and foundation conditions.

  b. A summary and description of static and dynamic properties of the soil and rock comprised by seismic Category I embankment dams and their foundations, natural and cut slopes, and all soil or rock slopes whose stability would directly or indirectly affect safety-related and seismic Category I facilities.

c. A summary and description of ground water, seepage, and high and low ground water conditions.

- Design Criteria and Analyses: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of design criteria and analyses is acceptable if it describes the criteria for the stability and design of all seismic Category I slopes and if valid static and dynamic analyses demonstrate that there is an adequate margin of safety.

- Boring Logs: In meeting the requirements of 10 CFR Parts 50 and 100, the applicant should describe the borings and soil testing carried out for slope stability studies and dam and dike analyses.

- Compacted Fill: In meeting the requirements of 10 CFR Part 50, the applicant should describe the excavation, backfill, and borrow material planned for any dams, dikes, and embankment slopes.

In addition, the geologic characteristics should be consistent with appropriate sections from the following:

- RG 1.27
- RG 1.28
- RG 1.132
- RG 1.138
- RG 1.198
- RG 1.206

### *2.5S.5.4 Technical Evaluation*

The staff reviewed the information in FSAR Section 2.5S.5:

*COL License Information Items*

- COL License Information Item 2.40 Stability of Slopes

- COL License Information Item 2.41 Embankments and Dams

The staff reviewed the resolution to the COL specific items related to the stability of all earth and rock slopes both natural and manmade (cuts, fill, embankments, dams, etc.) whose failure, under any conditions to which it could be exposed during the life of the plant, could adversely affect the safety of the plant, as included under Section 2.5S.5. of the STP COL FSAR.

With respect to COL License Information Items 2.40 and 2.41, the applicant stated that there are no soil or rock slopes or embankments and dams whose failure could adversely affect the safety-related structures at the STP site. The applicant referenced stability analyses of the main cooling reservoir embankment that were performed during the construction of STP, Units 1 and 2. The calculated factors of safety for various loading conditions are in FSAR Section 2.5S.5 and are summarized in SER Table 2.5S.5-1. The applicant also considered permanent deformation of the nonsafety-related main cooling reservoir embankment for an SSE with a peak ground acceleration of 0.1 g using a Newmark-type sliding block analysis. The applicant concluded that the main cooling reservoir is safe based on the computed factors of safety and on the distance of the STP, Units 3 and 4, power block area from the main cooling reservoir embankment—approximately 457 m (1,500 ft)—that would prevent a slope failure from impacting the Seismic Category 1 structures.

The staff reviewed the analyses and the calculated factors of safety and concluded that the factors of safety for the various loading conditions are satisfactory. The height of the main cooling reservoir dike closest to the STP, Unit 3 and 4, power blocks is approximately 10.7 m to 12.2 m (35 ft to 40 ft). The distance from the closest Seismic Category 1 structure is approximately 457 m (1,500 ft). The staff concluded that the separation between the slopes and the closest Category 1 structure is more than sufficient to preclude a potential slope failure from impacting any Category 1 structures. Furthermore, the staff observed no deformation along the crest or the slopes during a site visit, which indicates that the long-term stability of the main cooling reservoir embankment is satisfactory. The dynamic analysis for the main cooling reservoir embankment slopes used a peak ground acceleration of 0.1 g, which is the same acceleration as that used for STP, Units 3 and 4. The staff found that the factors of safety determined by the applicant's static and dynamic analyses performed for STP, Units 1 and 2, meet the standards set for STP, Units 3 and 4. The staff concluded that the applicant has sufficiently addressed COL License Information Items 2.40 and 2.41.

#### 2.5S.5.4.1 Slope Characteristics

The applicant describes in detail in the STP, Units 1 and 2, UFSAR the characteristics and stability analysis of the permanent main cooling reservoir embankment slopes. During the site audits, the staff examined the existing slopes at the site to confirm the slope locations, with respect to the seismic Category 1 structures and the lines and grades of the existing embankment. The staff also reviewed site boring logs and the site subsurface soil profile and determined that the main cooling reservoir embankment slopes are located a sufficient distance from the safety-related structures. Therefore, a slope failure will not adversely affect the safety of the structures.

The applicant also referred to a flood analysis based on a postulated 610-m (2,000-foot) breach of the dam and the potential impact on the safety-related structures. The staff's evaluation of this information is in Subsection 2.4S.4.4 of this SER. Based on these findings, the staff concluded that no slope failure at the site will adversely affect the safety of the nuclear power plant structures.

#### 2.5S.5.4.2 Design Criteria and Analysis

The staff reviewed the design criteria, especially the conditions that the applicant considered in assessing the factors of safety for slopes in the STP site area. The staff concluded that the applicant's factors of safety for the slopes in the STP site area are adequate.

#### 2.5S.5.4.3 Boring Logs

The staff reviewed the boring logs in the STP, Units 1 and 2, UFSAR and the STP, Units 3 and 4, FSAR. The staff concluded that the boring logs are sufficient to characterize the slopes in the STP site area.

2.5S.5.4.4 Compacted Fill

The staff's evaluation of compacted fill is in Subsection 2.5S.4.4, of this SER.

### *2.5S.5.5 Post Combined License Activities*

There are no post COL activities related to this section.

### *2.5S.5.6 Conclusion*

As discussed above, the applicant presented and substantiated information that established the stability of all earth and rock slopes, both natural and manmade, at the plant site. The staff reviewed the applicant's investigations of the slope stability studies and concluded that the margins of safety in the design analyses adequately demonstrate that natural and manmade slopes will remain stable under GMRS conditions, and safety-related earthwork will function reliably at the site to justify the soil and rock characteristics used in the design. The staff further concluded that the design analyses contain adequate margins of safety for the construction and operation of the nuclear power plant that meet the requirements of 10 CFR Part 50, Appendix A (GDC 1, 2, and 44); Appendices B and S of 10 CFR Part 50; and 10 CFR 100.23. The design analyses address COL License Information Items 2.40 and 2.41.

The staff's review also confirmed that the applicant has addressed the relevant information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Therefore, the staff finds that the STP, Units 3 and 4, site is suitable with respect to the criteria governing the stability of slopes.