2.5 Geology, Seismology, and Geotechnical Engineering

Section 2.5S of the Final Safety Analysis Report (FSAR) describes geologic, seismic, and geotechnical engineering properties of the proposed South Texas Project (STP) combined license (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 Regulatory Guide (RG) 1.206, "Combined License Applications for Nuclear Power Plants," 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 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.

This Safety Evaluation Report (SER), written by NRC staff, is divided into five main parts (SER Sections 2.5.1 through 2.5.5) 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.

2.5.1 Basic Geologic and Seismic Information

2.5.1.1 *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.

2.5.1.2 *Summary of Application*

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to Title of the *Code of Federal Regulations* (10 CFR) Part 52. 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 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.

2.5.1.2.1 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 states that the site is located within the Coastal Prairie subsection of the Gulf Coastal Plain Physiographic Province. SER Figure 2.5.1-1 (reproduced from FSAR Figure 2.5S.1-1) 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 illustrates all of the physiographic provinces within the state of Texas and briefly describes each province and subprovince.

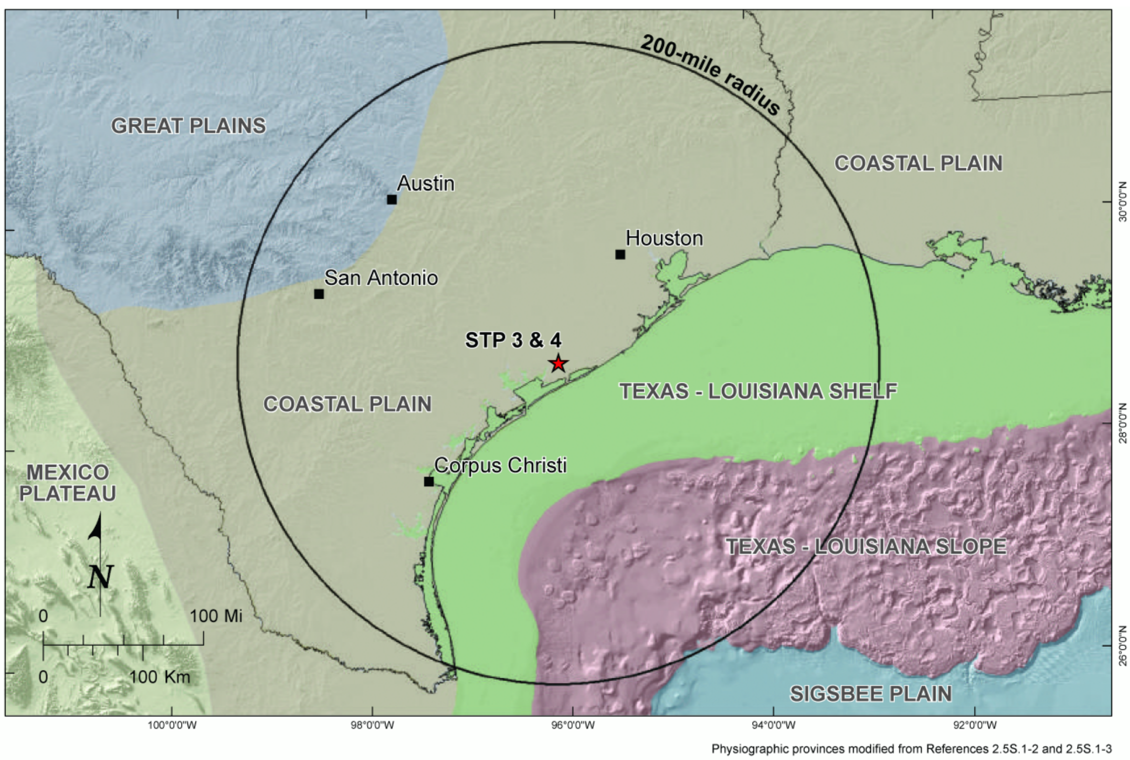


Figure 2.5.1-1. Map of Physiographic Provinces within the STP Site Region (FSAR Figure 2.5S.1-1)

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 southeasternmost 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 states 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 notes 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 states 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 2.5.1-2 (reproduced from FSAR figure 2.5S.1-5) is a geologic map of the STP site region showing the limited Paleozoic exposures (in purple).

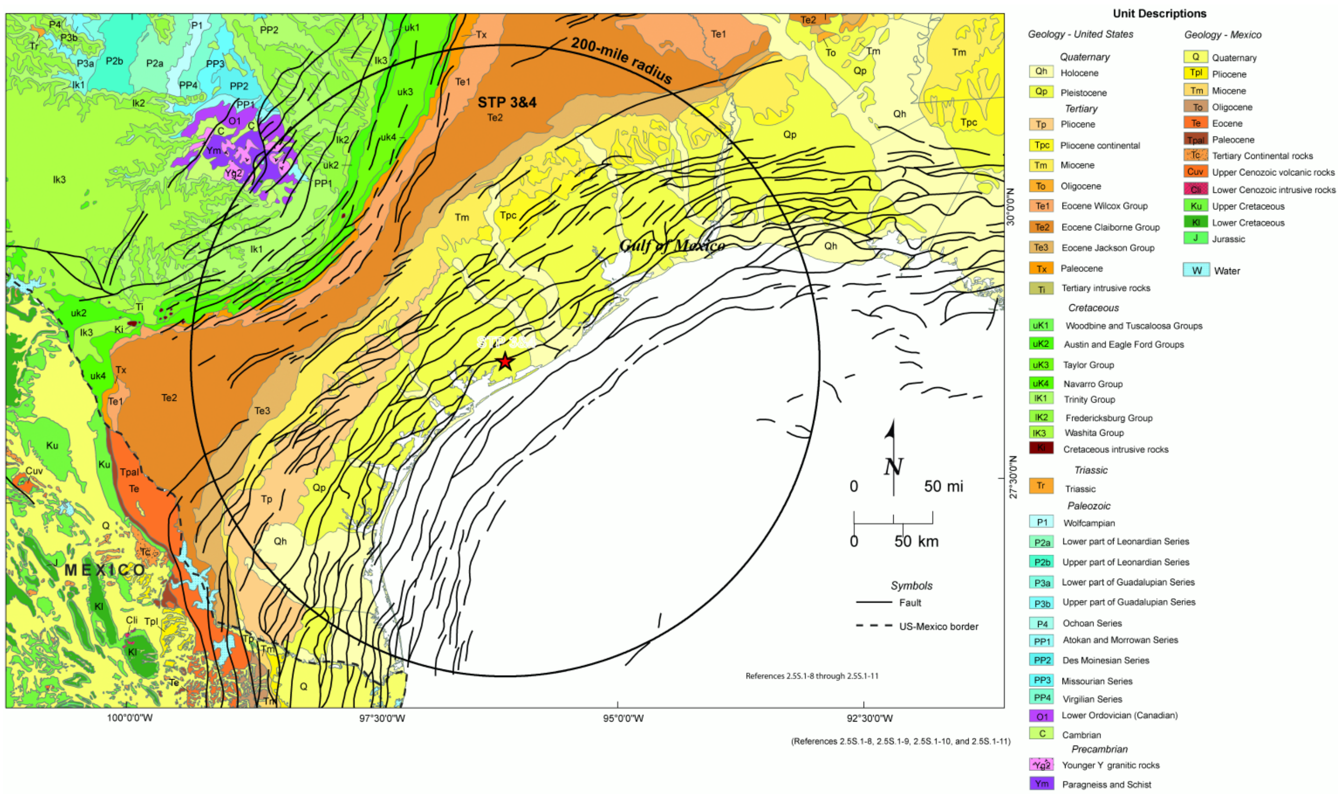


Figure 2.5.1-2. Geologic Map of the STP Site Region (FSAR Figure 2.5S.1-5)

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 states 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 and 2.5S.1-11 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 2.5.1-2 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. 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 states 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 Section 2.5S.1.2. The applicant states 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 Sections 2.5S.1.2 and 2.5S.3 and Subsection 2.5S.1.1.4.4.4.

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 states 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 concludes 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 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 states 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 states 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 states 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 states that (1) information reported since the 1986 EPRI study 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 concludes 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 reviews 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 and colleagues (USGS, 2002a, 2002b) and Keller (GSA, 1999). 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 states 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 and 2.5S.1-16. 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.

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. SER Figure 2.5.1-3 (reproduced from FSAR Figure 2.5S.1-17) shows all of the known tectonic features in the STP site region. The applicant concludes 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.

Mesozoic Tectonic Structures

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.

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 states 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.

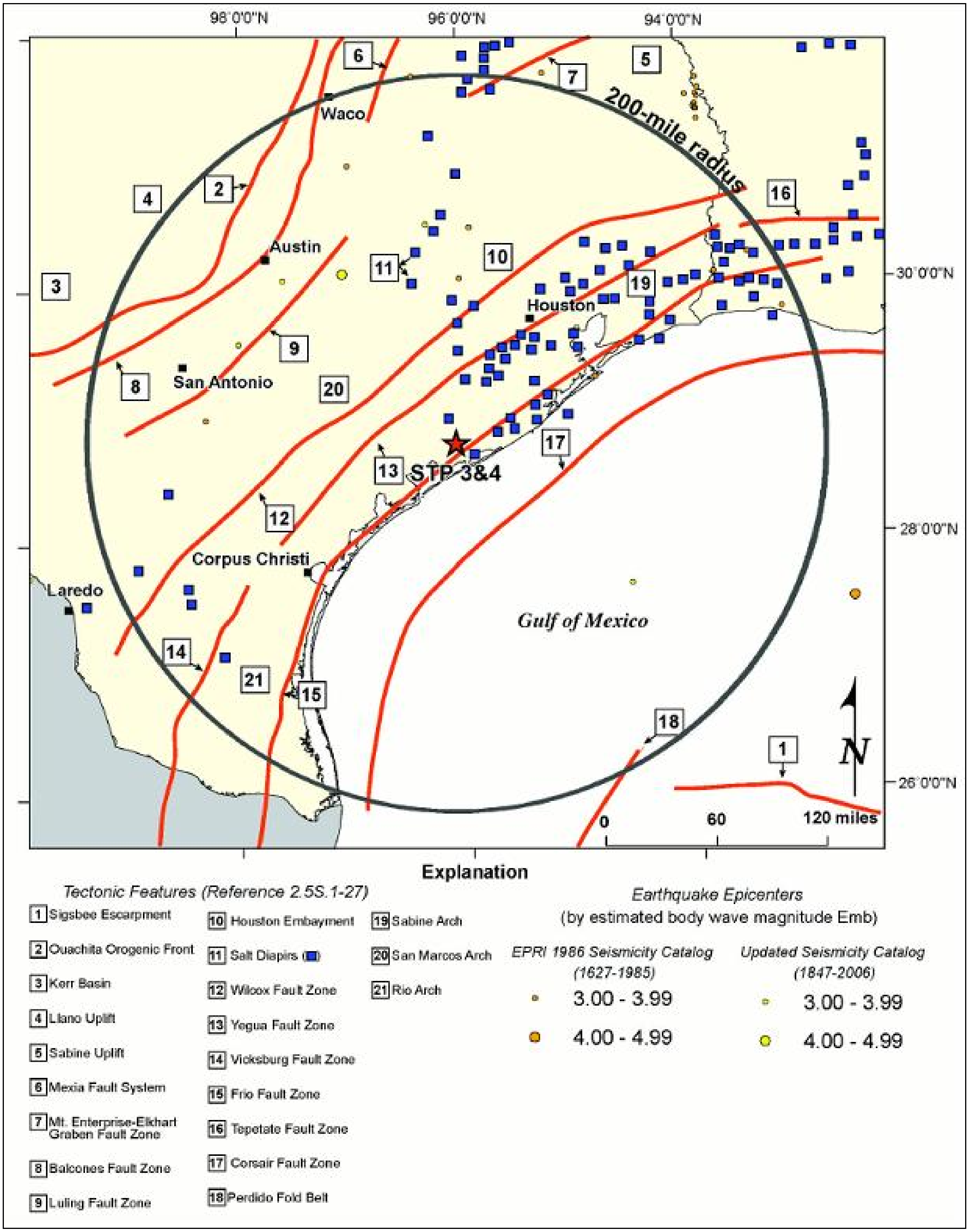


Figure 2.5.1-3. Tectonic Figures in the STP Site Region (FSAR Figure 2.5S.1-17)

Tertiary Tectonic Structures

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 notes 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 states 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 notes 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 states 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 2.5.1-3 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 states that neither the 1986 EPRI seismic source model studies nor 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 concludes 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 states 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 states 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 states 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 concludes 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 states 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 states that the potential features discussed by Collins and colleagues do not provide a sufficient basis to categorize this fault zone as a capable tectonic structure. In addition, the applicant states 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 concludes 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, includes this reduction in recurrence interval from 1,000 to 500 years.

Quaternary Growth Faults

The applicant states 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 concludes that gulf coastal growth faults are adequately accounted for in the EPRI seismic source models and no updates are required.

2.5.1.2.2 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 states that the topographic relief across the site is generally less than 4.6 m (15 ft).

Site Area Geologic History

The applicant describes 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 states 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 describes 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 notes 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 states 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 the1986 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.

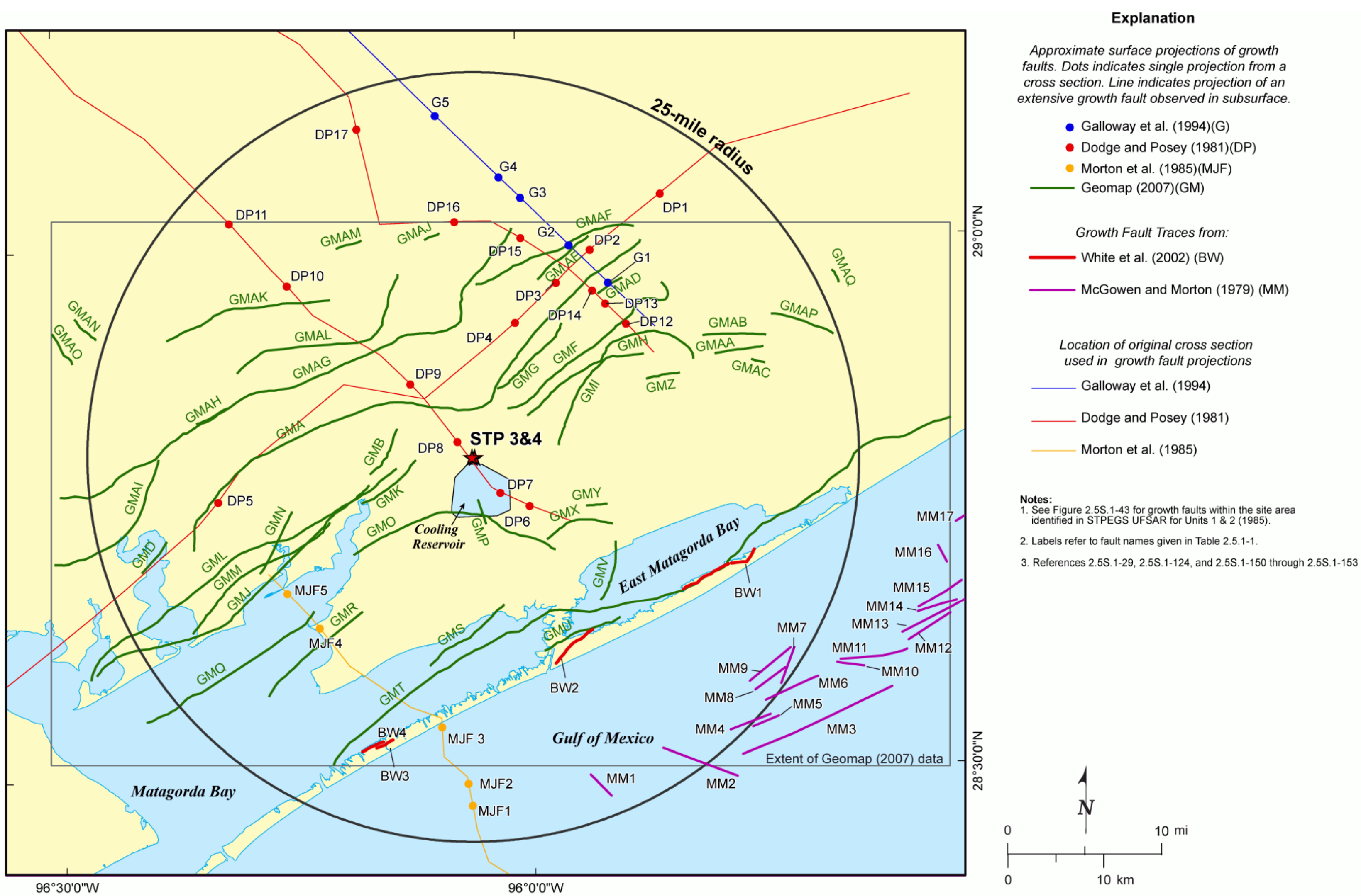
<u>Previous Growth Fault Studies in the Site Area</u>

The initial investigations for 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.

<u>Updated Information on Growth Faults in the Site Area</u>

The applicant compiled data from seven sources as part of the growth fault investigation for Units 3 and 4, including the UFSAR for Units 1 and 2. FSAR Table 2.5S.1-1 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 states that most of the faults described in the studies and depicted in SER Figure 2.5.1-4 (based on FSAR Figure 2.5S.1-42) can be projected to the surface. FSAR Figure 2.5S.1-43 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 notes 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.

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



25-mile radius
STP 3&4
Cooling Reservoir
East Matagorda Bay
Gulf of Mexico
Matagorda Bay
Extent of Geomap (2007) data
96°30'0"W
96°0'0"W
29°0'0"N
28°30'0"N
G1
G2
G3
G4
G5
DP1
DP2
DP3
DP4
DP5
DP6
DP7
DP8
DP9
DP10
DP11
DP12
DP13
DP14
DP15
DP16
DP17
MJF1
MJF2
MJF 3
MJF4
MJF5
BW1
BW2
BW3
BW4
MM1
MM2
MM3
MM4
MM5
MM6
MM7
MM8
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MM10
MM11
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GMP
GMQ
GMR
GMS
GMT
GMU
GMV
GMX
GMY
GMZ
GMAA
GMAB
GMAC
GMAD
GMAE
GMAF
GMAG
GMAH
GMAI
GMAJ
GMAK
GMAL
GMAM
GMAN
GMAO
GMAP
GMAQ
Explanation
Approximate surface projections of growth faults. Dots indicates single projection from a cross section. Line indicates projection of an extensive growth fault observed in subsurface.
Galloway et al. (1994)(G)
Dodge and Posey (1981)(DP)
Morton et al. (1985)(MJF)
Geomap (2007)(GM)
Growth Fault Traces from:
White et al. (2002) (BW)
McGowen and Morton (1979) (MM)
Location of original cross section used in growth fault projections
Galloway et al. (1994)
Dodge and Posey (1981)
Morton et al. (1985)
Notes:
1. See Figure 2.5S.1-43 for growth faults within the site area identified in STPEGS UFSAR for Units 1 & 2 (1985).
2. Labels refer to fault names given in Table 2.5.1-1.
3. References 2.5S.1-29, 2.5S.1-124, and 2.5S.1-150 through 2.5S.1-153
N
0
10 mi
0
10 km

Figure 2.5.1-4. Map of Growth Faults and Growth Fault Surface Projections Within the STP Site Vicinity (FSAR Figure 2.5S.1-42)

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 concludes 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 2.5.1-5 (reproduced from FSAR Figure 2.5S.1-45) 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. 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 states 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 concludes that there is no surficial evidence associated with this fault to suggest recent deformation.

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 concludes 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 discusses the engineering soil properties and the behavior of foundation materials in FSAR Section 2.5S.4. The applicant concludes 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 states 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 discusses construction dewatering in FSAR Section 2.5S.4, as it relates to geotechnical engineering. The applicant states that no mining or “excessive” ground water injection takes place in the STP site area. FSAR Section 2.4S.12 discusses ground water conditions and the effects of human activities related to ground water in more detail.

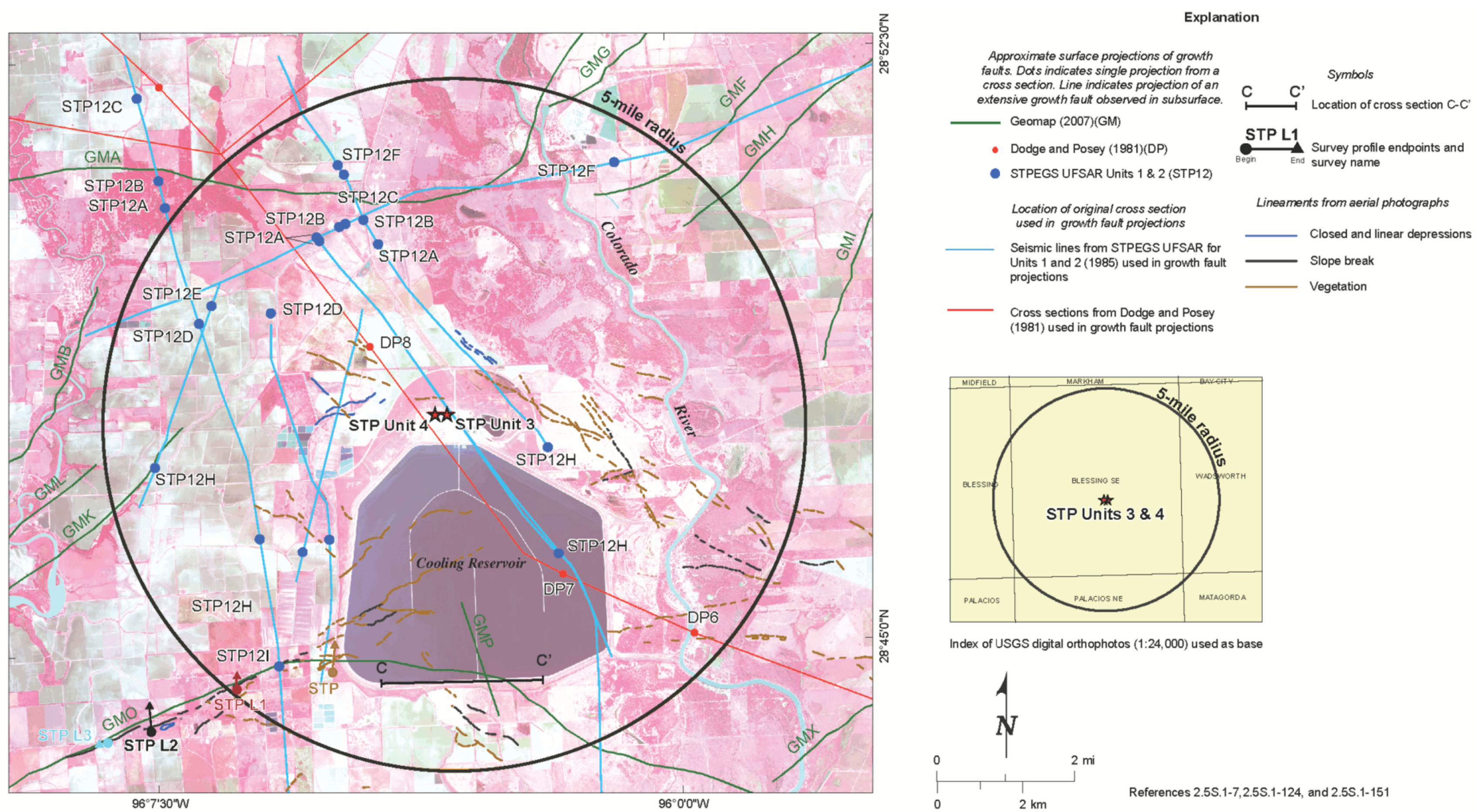

Figure 2.5.1-5. Map of Growth Fault Projections, Lineaments, and Topographic Survey Points Within the STP Site Area (Reproduced from FSAR Figure 2.5S.1-45)

2.5.1.3 *Regulatory Basis*

The regulatory basis and acceptance criteria for reviewing COL License Information Item 2.23 are in Section 2.5.1 of NUREG–0800. The applicable regulatory requirements for reviewing geologic and seismic information are as follows:

(1) 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.

(2) 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 acceptance criteria in Section 2.5.1 of NUREG–0800 are as follows:

(1) <u>Regional Geology</u>: 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.

(2) <u>Site Geology</u>: In meeting the requirements of 10 CFR 52.79 and 10 CFR 100.23 and the regulatory positions in RGs 1.208, 1.132, 1.138, 1.198, 1.206, and 4.7, FSAR Subsection 2.5S.1.2 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.

2.5.1.4 *Technical Evaluation*

NRC staff reviewed the information in Section 2.5S.1 of the STP Units 3 and 4 COL FSAR:

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

- COL License Information Item 2.23 — Geology, Seismology, and Geotechnical Engineering

NRC staff reviewed the information provided by the applicant in COL FSAR Section 2.5S.1 of the STP COL application 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 of 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 is presented below.

2.5.1.4.1 Regional Geology

The NRC staff's review focused on STP 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 states 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 notes 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

NRC 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 COL application and in accordance with RG 1.208.

Regional Stratigraphy

The NRC 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. The applicant states 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 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. The applicant's response to **RAI 02.05.01-16** dated July 9, 2008 (ML081960070), thoroughly describes 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.

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 concludes 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.

NRC 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 and 2.5S.1-20. 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. In response to these RAIs, the applicant provides all corrections and clarifications. Therefore, **RAI 02.05.01-2**, **RAI 02.05.01-5**, and **RAI 02.05.01-17** are resolved.

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. NRC 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 the response to **RAI 02.05.01-1** dated July 24, 2008 (ML082100162), the applicant provides a more thorough description of Pleistocene and Holocene sediments of the Quaternary Period based on the recent literature. The applicant discusses 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 states 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.

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.

<u>Regional Tectonic Stress.</u> In STP COL FSAR Subsection 2.5S.1.1.4.2, the applicant states 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 states 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. NRC 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.

<u>Regional Gravity and Magnetic Data.</u> The NRC 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 STP Col application. The applicant states that gravity and magnetic anomalies identified in the available data correspond with major tectonic events discussed in the FSAR. In addition, the applicant identifies 10 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 COL application.

<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. The applicant concludes 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. NRC 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.

The applicant's response to **RAI 02.05.01-3** dated August 27, 2008 (ML082490086), explains that the seismic hazard in the STP site region is modeled using areal source zones determined by the EPRI-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 states 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 elaborates on these investigations in FSAR Sections 2.5S.1 and 2.5S.2. The applicant describes 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 states 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 identifies 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. **RAI 02.05.01-3** is resolved.

<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 states 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 concludes that no salt structures are identified within the immediate STP site. NRC 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 states 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 notes 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 NRC 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 suggests 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 concludes 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 concludes 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 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 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. NRC 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.

The applicant's response to **RAI 02.05.01-4** dated September 4, 2008 (ML082530449), discusses 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 states 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 states 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 4mm/yr (0.17 inches/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 inches/yr) across the MEEG system. Therefore, **RAI 02.05.01-4** is resolved.

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 states 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 concludes 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, NRC 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 the response to **RAI 02.05.01-6** dated July 16, 2008 (ML082030326), the applicant states 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 points out that the Collins et al. (1990) study was published after the EPRI-SOG study and was therefore evaluated for the STP 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 states 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 states 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 states 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 2.5.2.4 of this SER includes the staff's evaluation of the seismic source characterizations that the applicant incorporated into its PSHA for the STP site. **RAI 02.05.01-6** is resolved.

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 1 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 includes 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, NRC 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 2.5.2 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 states 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 implies that these faults do not have the ability to generate significant earthquakes. The applicant concludes that gulf coastal growth faults are adequately accounted for in the EPRI seismic source models and no updates are required.

NRC 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 provides 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

NRC staff reviewed STP COL 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 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

NRC staff reviewed STP COL 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.

2.5.1.4.2 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 SER subsections discuss the NRC 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 2.5.1-6 (reproduced from FSAR Figure 2.5S.1-27), 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. NRC 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.

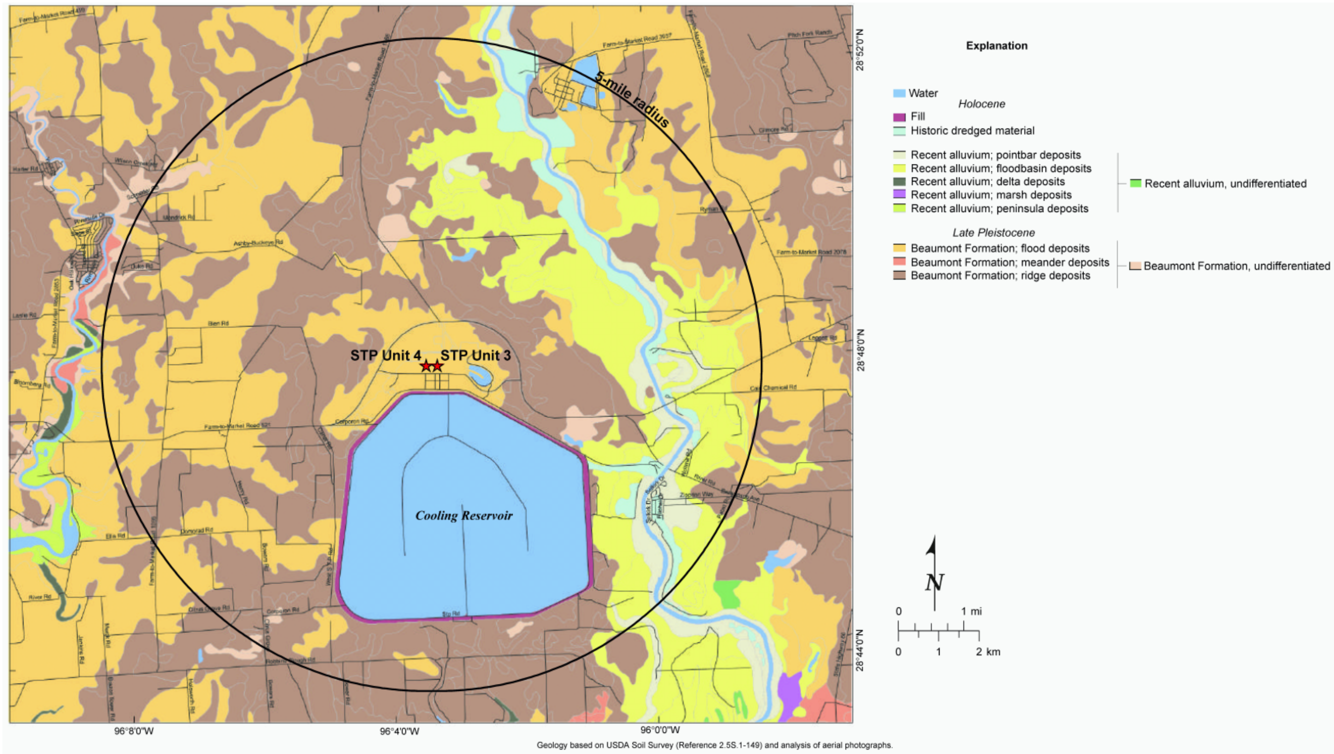

Figure 2.5.1-6. Geologic Map of the STP Site Area (FSAR Figure 2.5S.1-27)

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 states that the Quaternary Period was dominated by an almost continuous sedimentary deposition that led to the gulfward migration of the shoreline. NRC 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 states 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 NRC 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 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 concludes that no new information developed about such structures has emerged since the 1986 EPRI studies. The applicant states that buried growth faults are the only geologic structures in the STP site area. However, the applicant concludes 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 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 states 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 states, 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.

NRC 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. The applicant's response to **RAI 02.05.01-8** dated July 16, 2008 (ML082030326), updates FSAR Figures 2.5S.1-44 and 2.5S.1-45 to include the locations of all "anomalous geomorphic features" that might indicate surficial deformation due to growth faulting. The applicant states 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 states 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.

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. The applicant's response to **RAI 02.05.01-7** dated October 1, 2008 (ML082770138), provides 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 describes 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. **RAI 02.05.01-7** and **RAI 02.05.01-8** are resolved.

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 STP12I (Matagorda GMO); and (2) to discuss the uncertainties in projecting this growth fault to the surface. The applicant's response to **RAI 02.05.01-9** dated October 1, 2008, states that the locations of the topographic surveys (as shown in SER Figure 2.5.1-5) 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 states that even though there are uncertainties in the projection of growth fault Matagorda STP12I (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 STP12I (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 STP12I (Matagorda GMO) bends to the southeast around the cooling reservoir (as shown in SER Figures 2.5.1-4 and 2.5.1-5) 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.

The applicant's response to **RAI 2.5.1-10** dated September 4, 2008 (ML082530449), states 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 12I, the applicant states that the methods used to measure structural relief across the projection of growth fault Matagorda STP12I (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 concludes 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 notes 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 STP12I (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 2.5.1-5 (based on FSAR Figure 2.5S.1-45) shows linear features within the cooling reservoir that represent slope breaks and vegetation lineaments along strike with growth fault Matagorda STP12I (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 STP12I (Matagorda GMO), which is different from the projection shown in SER Figures 2.5.1-4 and 2.5.1-5.

The applicant's response to **RAI 02.05.01-20** dated July 20, 2009 (ML092030132), states 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 concludes 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 STPI21, 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 2.5.1-5 (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.

The staff noted that FSAR Figure 2.5S.1-43 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.

The applicant's response to **RAI 02.05.01-13** dated August 12, 2008 (ML082270381), provides 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 states 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.

The applicant's response to **RAI 02.05.01-21** dated July 20, 2009 (ML092030132), provides 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 states that the UFSAR for STP Units 1 and 2 does not discuss resolution limits associated with the seismic reflection data. However, the seismic reflection data identify two growth faults, STP12A and STP12I, which approach within 275 m (900 ft) of the earth's surface beneath the STP site area. The applicant also states that shallow growth faults typically "sole into deeper growth fault systems." Therefore, the applicant concludes 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 notes 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 concludes 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 states that the only evidence of growth faulting in the STP site area is “broad, monoclinal folding and tilting” associated with growth fault Matagorda STP12I and Matagorda GMO, as previously described. The applicant also refers to the response to **RAI 02.05.01-10,** which concludes 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.

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. The applicant’s response to **RAI 02.05.01-19** dated July 20, 2009 (ML092030132), states that the surface projection of growth fault GMP (as indicated in SER Figure 2.5.1-5) appears to trend northwest through the cooling reservoir and just to the west of the proposed STP Units 3 and 4. The applicant states 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 states 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 concludes 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. The applicant’s response to **RAI 02.05.02-22** dated

March 10, 2010 (ML100620824), states 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 response to **RAI 02.05.01-22**, the applicant plans to modify FSAR Subsection 2.5S.1.2.4.3 in a future revision to “include a discussion of how the Geomap data demonstrates the change in the strike of GMP.” In its response, the applicant indicates 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 states 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 proposes a revision to FSAR Subsection 2.5S.1.2.4.3 which supersedes the applicant’s proposed revision in its response to **RAI 02.05.01-19**. Therefore, the staff concluded that **RAI 02.05.01-19** is resolved and that the applicant’s proposed revision to FSAR Subsection 2.5S.1.2.4.3 in response to **RAI 02.05.01-22** is being tracked as **Confirmatory Item 02.05.01-1**.

In addition to the RAIs described above, the staff issued **RAI 02.05.01-11** and **RAI 02.05.01-12** requesting the applicant to clarify the information in the FSAR regarding growth faulting, in order to complete the evaluation of the applicant’s information. The applicant provided the clarifying information, which the staff found acceptable. Therefore, RAI 02.05.01-11 and RAI 02.05.01-12 are resolved.

Staff Conclusions Regarding Site Area Geologic Structures

NRC staff reviewed FSAR Subsection 2.5S.1.2.4.1 and concluded that pending the resolution of **Confirmatory Item 02.05.01-1**, 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 pending the resolution of **Confirmatory Item 02.05.01-1**, 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 concludes that there is no evidence for dissolution, zones of deformation, or volcanic activity in the STP site area. NRC 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 discusses 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.1.4.2.

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 states that FSAR Section 2.5S.4 discusses engineering soil properties and the behavior of foundation materials. The applicant concludes 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.

<u>Prior Earthquake Effects.</u> In FSAR Subsection 2.5S.1.2.6.4, the applicant states 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. NRC 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. NRC 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.

The applicant's response to **RAI 02.05.01-15** dated July 16, 2008 (ML082030326), identifies an extensive literature review for the STP 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 states 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 discusses 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 concludes 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, 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. The applicant's response to **RAI 02.5.01-018** dated July 20, 2009 (ML092030132), provides 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 states that most of the riverbank that the applicant investigated along the Colorado River provided good exposure to look for sedimentary features. The applicant states 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. The applicant's response to **RAI 02.05.01-18** also notes that FSAR Subsection 2.5S.1.2.6.4 will be updated with a more detailed description of the investigations for prior earthquake effects at the STP site in a future FSAR revision. Verification of the proposed commitment in the revised FSAR is being tracked as **Confirmatory Item 02.05.01-2**.

<u>Effects of Human Activities.</u> 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 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 states that no mining or "excessive" ground water injection takes place in the STP site area. The applicant discusses ground water conditions and the effects of human activities related to ground water in more detail in FSAR Section 2.4S.12. The NRC 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. The applicant's response to **RAI 02.05.01-14** dated August 27, 2008 (ML082490086), states that growth fault Matagorda GMO (STP12I) 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 describes fluid withdrawal activities associated with the Chicot aquifer and hydrocarbon production activities near the STP site. The applicant states 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 states 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) documents "no noticeable surface deformation from movement on growth fault GMO/STP12I for at least the last 50 years." Given the evidence presented above, the applicant concludes 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.

<u>Staff Conclusions Regarding the Site Engineering Geology Evaluation</u>. NRC staff reviewed FSAR Subsection 2.5S.1.2.6 and concluded that pending verification of **Confirmatory Item 02.05.01-2**, the applicant has provided an adequate description of the site engineering geology for the STP site. The applicant states 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. The applicant's response to **RAI 02.05.01-23** dated March 1, 2010 (ML100620824), states that the Section 3.9S.3.11 of the Environmental Report (in COL application Part 3) describes the applicant's plans for geologic mapping of excavations. In addition, the applicant states that it will provide a description of its plans for geologic mapping during excavations in a future revision of the FSAR. The applicant's

commitment to revise the FSAR is being tracked as **Confirmatory Item 02.05.01-3**. These actions are proposed **License Condition 2.5.1-1, as specified in SER Subsection 2.5.1.5**.

2.5.1.5 *Post Combined License Activities*

The following License Condition is identified in SER Subsection 2.5.1.4.2 as the responsibility of the COL Holder:

License Condition 2.5.1-1: The applicant must perform geologic mapping of future excavations for safety-related structures; evaluate any geologic features discovered in the excavations; and notify the NRC once excavations for safety-related structures are open for examination by the NRC staff.

2.5.1.6 *Conclusion*

NRC 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 expected to be addressed in the STP COL FSAR related to this subsection. With the exceptions of the **confirmatory items** identified above, no outstanding information is expected to be addressed in the COL FSAR related to this section.

As a result of the confirmatory items, the staff was unable to finalize the conclusion relating to the basic geologic and seismic information, in accordance with the requirements of 10 CFR 52.79 and 10 CFR 100.23 and the guidance in RG 1.208.

2.5.2 Vibratory Ground Motion

2.5.2.1 *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 identifies and characterizes 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.

2.5.2.2 *Summary of Application*

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52. The applicant identifies no departures from the certified design and provides supplemental, site-specific information to address COL License Information Item 2.24.

COL License Information Item

- 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.

2.5.2.2.1 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. The updated seismicity within the Gulf of Mexico is listed in FSAR Table 2.5S.2-4. SER Figure 2.5.2-1 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 2.5.2-1, 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 2.5.2-1. 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.

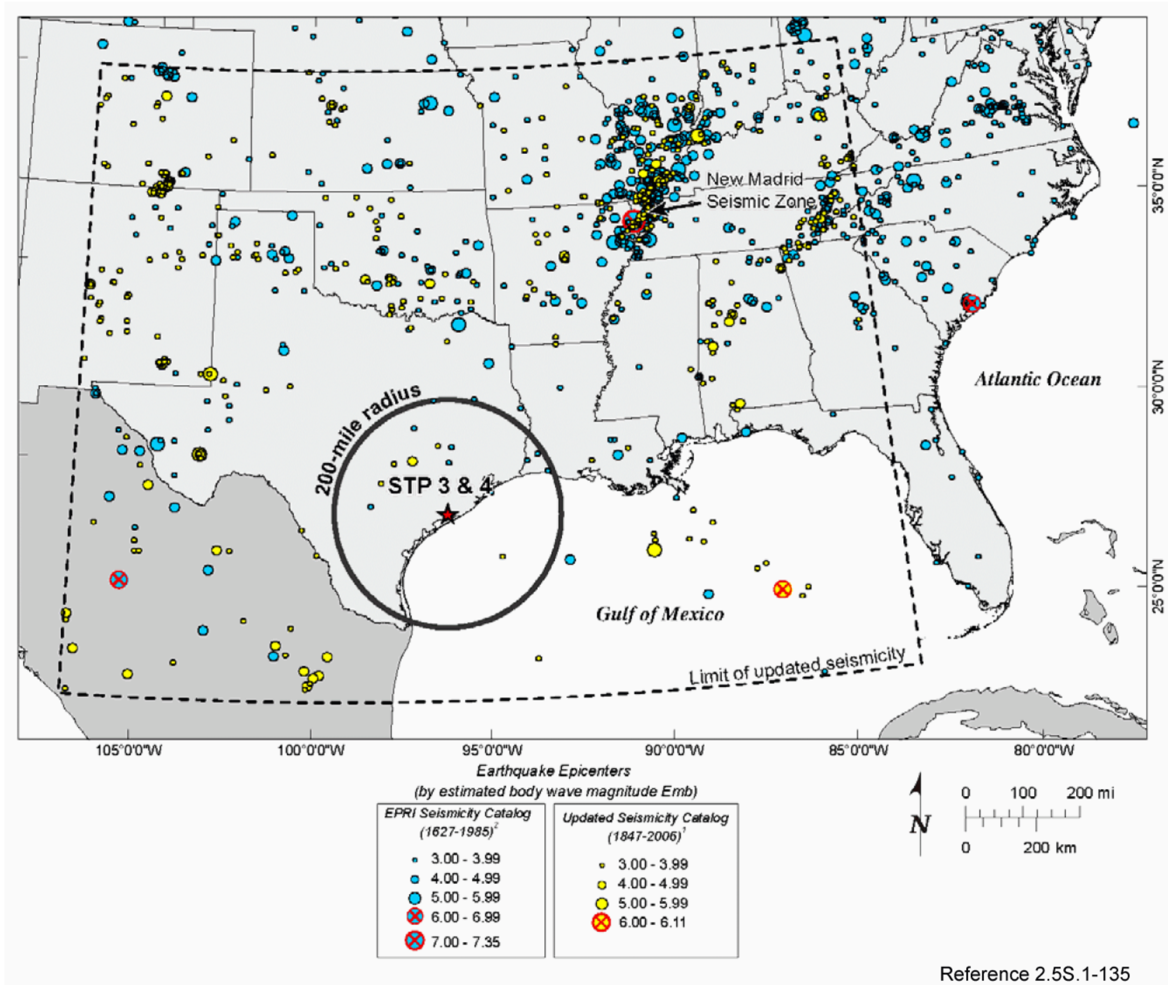

Figure 2.5S.1-26 New Madrid Seismic Zone

Figure 2.5.2-1. 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 concludes 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.

2.5.2.2.2 Geologic and Tectonic Characteristics of the Site and Region

FSAR Subsection 2.5S.2.2 describes the original EPRI-SOG (EPRI NP-4726) 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 1989 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 2.5.2.2.4 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 2.5.2-1).

<u>Bechtel Group</u>. 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 states 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.

2.5.2.2.3 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 2.5.2.2.4 describes the applicant's updated data in more detail.

2.5.2.2.4 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 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 2.5.2-1, 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 (TR-1009684) ground motion prediction models instead of the 1989 EPRI-SOG (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 1009684) 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 2.5.2-1 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 2.5.2-1 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 2.5.2-1 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 2.5.2-1. 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] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	6.1 [0.1] 6.4 [0.4] 6.6 [0.5]
	BZ2	Texas Platform	0.1	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	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]
Dames & Moore	20	South Coastal Margin	1.0	5.3 [0.8] 7.3 [0.2]	5.5 [0.8] 7.3 [0.2]
	25	Ouachitas Fold Belt	0.35	5.5 [0.8] 7.2 [0.2]	No update
	C08	Combination zone: 25 excluding 25A (Ouachitas Fold Belt excluding Kink in Fold Belt)	NA	5.5 [0.8] 7.2 [0.2]	No update
Law Engineering	124	New Mexico – Texas Block	1.0	4.9 [0.3] 5.5 [0.5] 5.8 [0.2]	No update
	126	South Coastal Block	1.0	4.6 [0.9] 4.9 [0.1]	5.5 [0.9] 5.7 [0.1]
Rondout Associates	51	Gulf Coast to Bahamas Fracture Zone	1.0	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	6.1 [0.3] 6.3 [0.55] 6.5 [0.15]
Weston Geophysical Corporation	107	Gulf Coast	1.0	5.4 [0.71] 6.0 [0.29]	6.6 [0.89] 7.2 [0.11]
Woodward-Clyde Consultants	B43	Central US Backgrounds	NA	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	No update

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 states that source zone B43 (Woodward-Clyde Consultants) is a sufficient distance from the epicenters of the recent earthquakes not to be updated. SER

Table 2.5.2-1 and FSAR Table 2.5S.2-13 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 states 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 states 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 1 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 (2006) resulting in modified aleatory uncertainties and weights. The 2006 EPRI study found that the aleatory uncertainties were too large in EPRI (2004), 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 2.5.2-1 and Exelon's (2006) updated NMSZ. 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 2.5.2-2 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.

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 2.5.2-2 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 2.5.2-2.

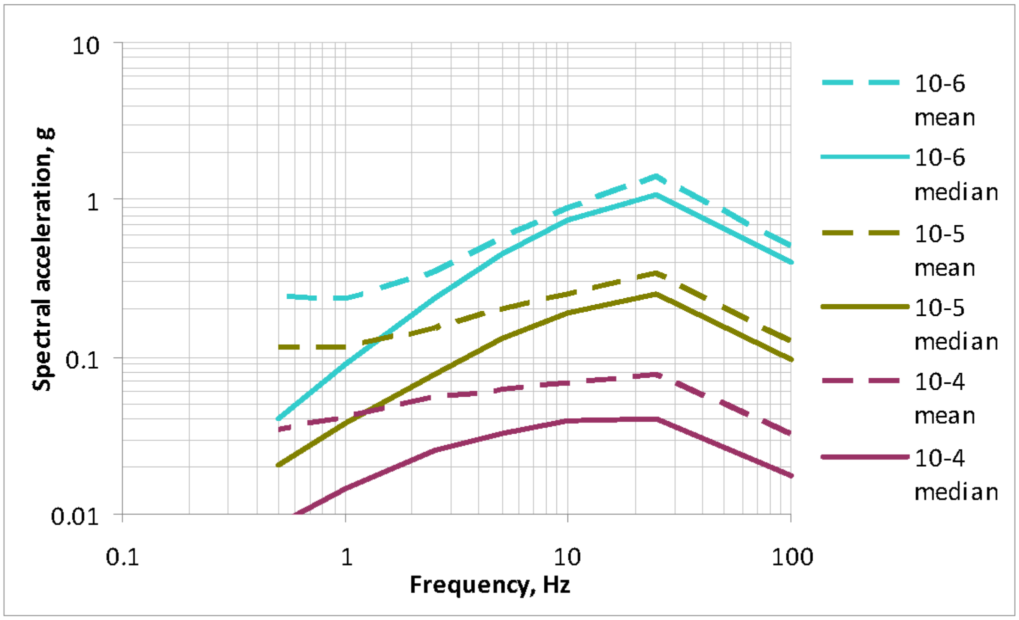

Figure 2.5.2-2. Mean and Median Rock Uniform Hazard Response Spectra (UHRS). (FSAR Figure 2.5S.2-26.)

Table 2.5.2-2. Controlling Earthquakes for Different Annual Frequencies of Exceedance and Structural Frequencies, Where M is Magnitude and R is Source-to-Site Distance. (The Applicant's Representative Controlling Earthquakes are Shaded in Gray) (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

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 2.5.2-2), using the controlling earthquake magnitude and distance values in SER Table 2.5.2-2 and the hard rock spectral shapes for CEUS earthquake ground motions recommended in NUREG/CR-6728. The resulting 10^{-4} and 10^{-5} smoothed spectra are shown in SER Figure 2.5.2-3.

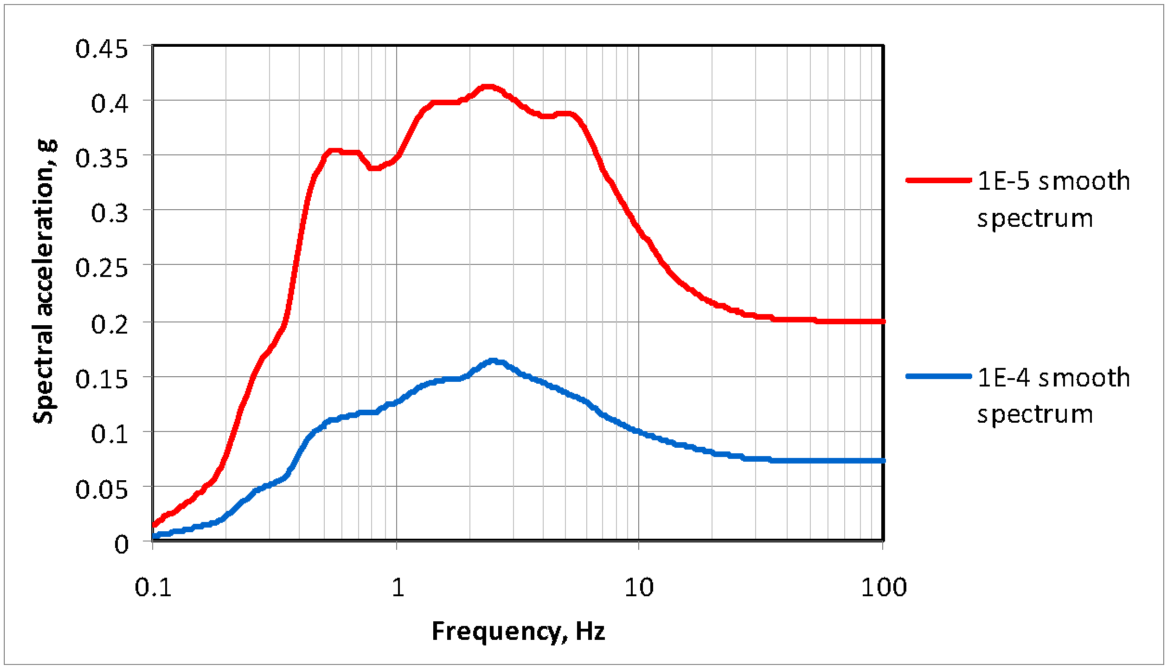

Figure 2.5.2-3. Smooth 10^{-4} and 10^{-5} rock UHRS (FSAR Figure 2.5S.2-51)

2.5.2.2.5 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 states 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 (1993; 2005). Layer damping is an assumed additive for soil layers and is dependant on the individual soil layers. The applicant used site-specific geophysical data (S-wave velocities [V_S]), the generic EPRI (1993) shear modulus, and damping ratio curves to determine a value for κ [in] 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 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. (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 and 2.5S.2-38, 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 preformed 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. 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 2.5.2-4).

2.5.2.2.6 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)/Safety Engineering Inspection (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 (2001). 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 2.5.2-4 and 2.5.2-5, respectively.

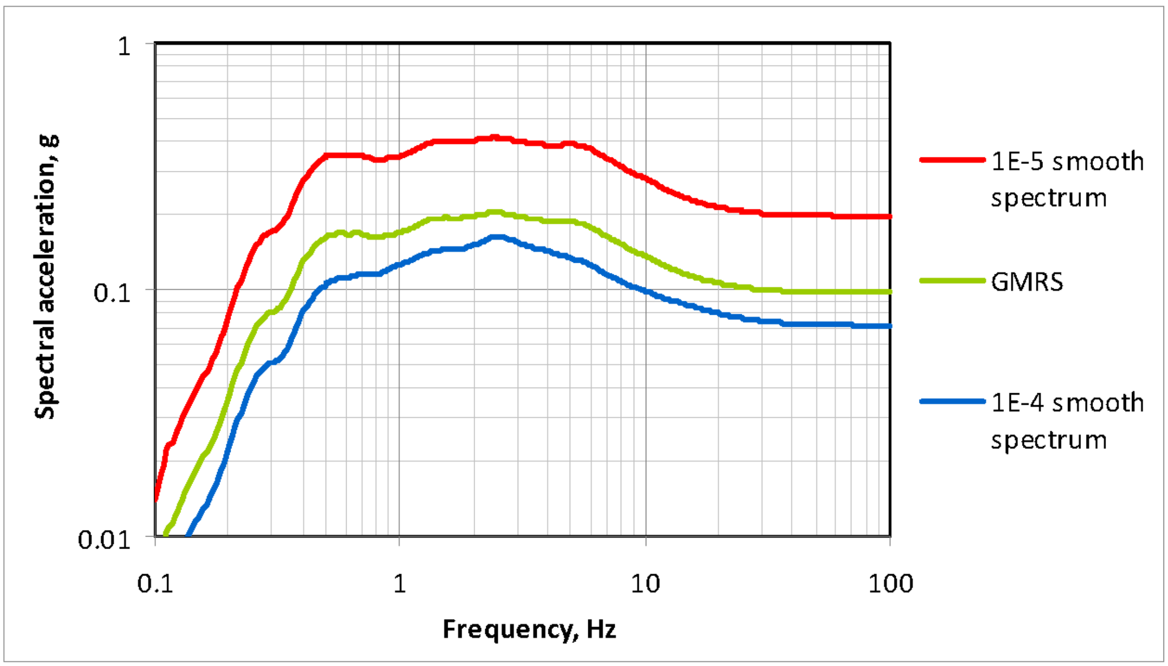

Figure 2.5.2-4. Smoothed 10^{-4} and 10^{-5} Ground Surface (Soil) Horizontal UHRS and Resulting Horizontal GMRS (FSAR Figure 2.5S.2-52)

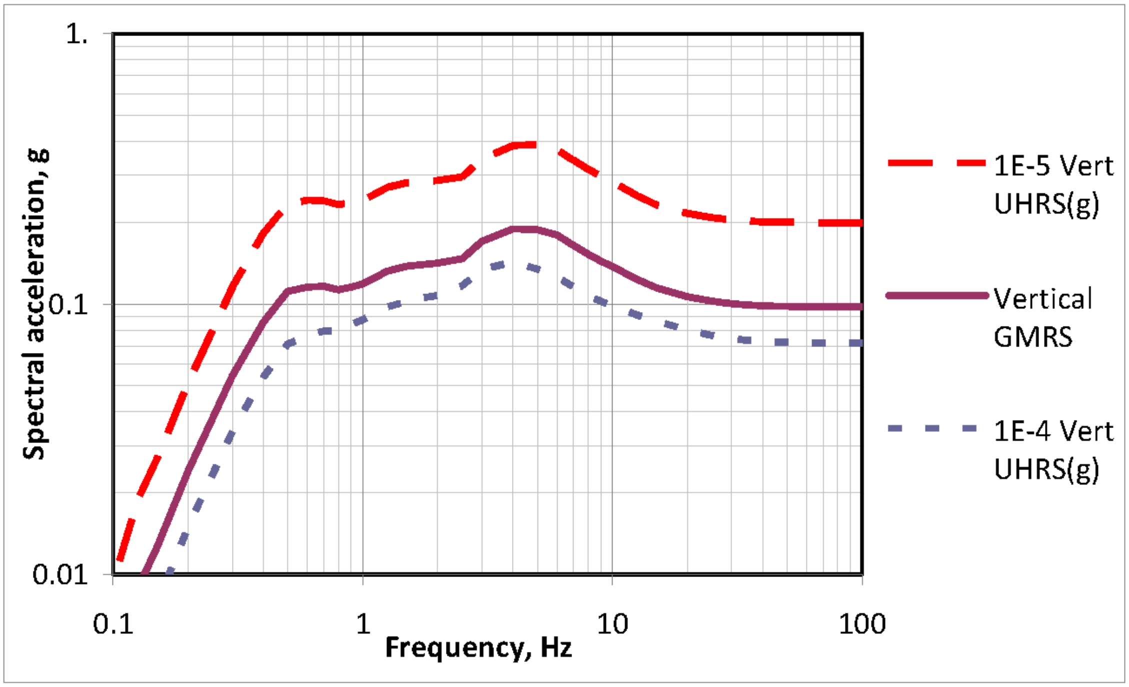

Figure 2.5.2-5. Vertical 10^{-4} and 10^{-5} Soil UHRS and Resulting Vertical GMRS. (Referred to in the figure as DRS)(FSAR Figure 2.5S.2-54)

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 2.5.2-4) 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 because they are conservative and simple when comparing the values obtained from other methods. SER Figure 2.5.2-5 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.

2.5.2.3 *Regulatory Basis*

The regulatory basis and acceptance criteria for reviewing COL License Information Item 2.23 are in Section 2.5.2 of NUREG–0800. The applicable regulatory requirements for reviewing the applicant's discussion of vibratory ground motion are the following:

(1) 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 RGs 1.132, "Site Investigations for Foundations of Nuclear Power Plants" and RG 1.208, "A Performance-Based Approach to Define Site-Specific Earthquake Ground Motion.".

(2) 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 related acceptance criteria are summarized from SRP Section 2.5.2:

(1) Seismicity: 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.

(2) Geologic and Tectonic Characteristics of Site and Region: 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.

(3) Correlation of Earthquake Activity with Seismic Sources: 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.

(4) Probabilistic Seismic Hazard Analysis and Controlling Earthquakes: 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.

(5) Seismic Wave Transmission Characteristics of the Site: In the PSHA procedure described in Regulatory Guide 1.165, the controlling earthquakes are determined for generic rock conditions.

(6) Ground Motion Response Spectra: 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, "Site Investigations for Foundations of Nuclear Power Plants"; RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion"; and RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)."

2.5.2.4 *Technical Evaluation*

NRC staff reviewed Section 2.5S.2 of the STP Units 3 and 4 COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the information in the COL represent the complete scope of information relating to this review topic. The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to the vibratory ground motion. The staff's technical evaluation of the information incorporated by reference related to the vibratory ground motion will be documented in the staff's SER on the DC application for the ABWR design.

NRC staff reviewed the Information Items in Section 2.5S.2 of the STP Units 3 and 4 COL FSAR:

- COL License Information Item 2.24 Vibratory Ground Motion

NRC 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 2.5.2.4 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 of 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.

2.5.2.4.1 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 of 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 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 describes the conversion of the different magnitude scales from each of the catalogs to m_b and the subsequent conversion to M. NRC 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. The applicant's response to **RAI 02.05.02-1** dated July 9, 2008 (ML081960070), includes a table showing how the uncertainty in the magnitude estimates varies with each of the different magnitudes. The applicant's response to **RAI 02.05.02-2** dated July 9, 2008, clarifies 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. RAI 02.05.02-1 and RAI 02.05.02-2 are resolved.

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.

NRC 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. The applicant's response to **RAI 02.05.02-3** dated July 9, 2008, provides 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 therefore adequately converted the Gulf earthquakes with the M_s scale to the m_b scale. Therefore, RAI 02.05.02-3 is resolved.

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, The applicant's response to **RAI 02.05.02-4** dated July 9, 2008, states that "MAIN" refers to independent events and "non-MAIN" refers to dependent events. The applicant then explains 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.

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. The applicant's response to **RAI 02.05.02-6** dated July 9, 2008, explains 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 states 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.

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. The applicant's response to **RAI 02.05.02-7** dated July 9, 2008, states 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 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. RAI 02.05.02 -7 is resolved.

Staff Conclusions Regarding Seismicity

NRC 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.

2.5.2.4.2 Geologic and Tectonic Characteristics of the Site and Region

This section of the safety evaluation includes the NRC staff's evaluation of the seismic source models the applicant uses as part of the PSHA for the STP site. The applicant describes 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, "A Performance-

Based Approach to Define the Site-Specific Earthquake Ground Motion," 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) Mmax 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 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 Mmax 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.

NRC 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." The applicant's response to **RAI 02.05.02-10** dated September 4, 2008 (ML082530449), updates this comparison of the USGS and the EPRI-SOG EST modeling approaches in FSAR Subsection 2.5S.2.2.8. The applicant also provides 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 describes 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.

The staff issued **RAI 02.05.02-5** asking whether the applicant had considered the more recent studies by Johnston (1994) and Kanter (1994) on worldwide earthquakes in SCRs as potential sources for updating the EPRI-SOG seismic source models. The applicant's response to **RAI 02.05.02-5** dated September 4, 2008, states 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 (1994) and Kanter (1994) studies do not constitute information that was not available to the EPRI-SOG ESTs. Therefore, RAI 02.05.02-5 is resolved.

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 2.5.2.4.1 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 4 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.

NRC 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." The applicant's response to **RAI 02.05.02-13** dated September 4, 2008 (ML092530449), states that the updated maximum magnitude distributions for the Gulf Coast seismic sources used an SSHAC Level 2 study. The response includes 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** provides 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. The applicant's response to **RAI 02.05.02-21** dated September 21, 2009 (ML092710096), provides 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, Mmax) 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 notes 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 states 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 2.5.2-6 shows the EPRI-SOG GCSZ geometries along with the epicenters for the two Gulf earthquakes.

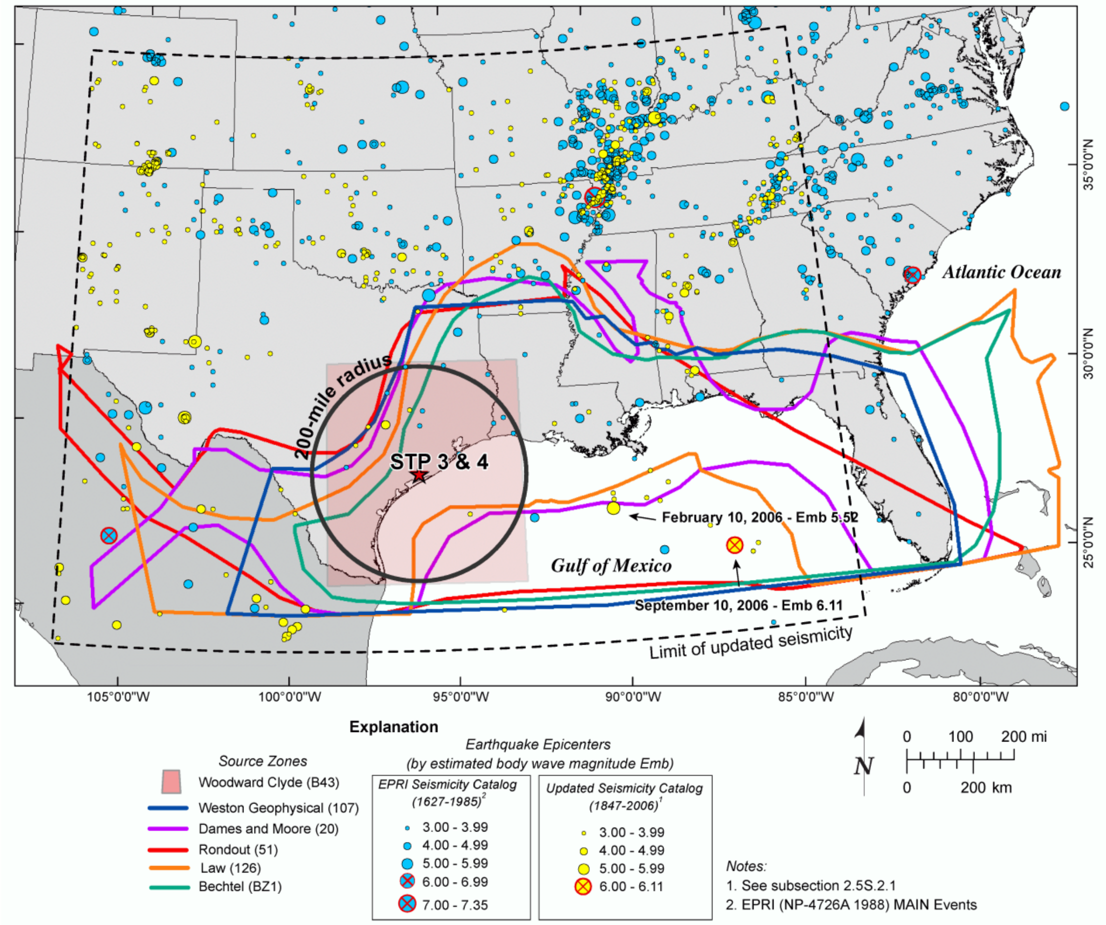

Figure 2.5.2-5. EPRI EST Gulf Coast Background Source Zones (FSAR Figure 2.5S.2-8)

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 states that the SSHAC peer review panel "did not think it was appropriate to base the updated Mmax 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 2.5.2-3. Updated Maximum Magnitude (Mmax) Distributions (m_b) for the Gulf Coast Seismic Source Zones

EST	ORIGINAL Mmax	TI TEAM INITIAL Mmax	FINAL Mmax
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

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 Mmax 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 Mmax value of m_b 7.2 is supported by the informed technical community.

To resolve the staff's concern with regard to the different Mmax 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 Mmax distributions into the GCSZ models. The applicant's sensitivity test covered three different Mmax update scenarios. Scenario 1 incorporated the TIs initial Mmax 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 Mmax 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 Mmax distribution for all six EPRI teams' models (see SER Table 2.5.2-4).

Table 2.5.2-4. Maximum magnitude distributions and scenarios used in 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] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	6.1 [0.10] 6.4 [0.40] 6.6 [0.50]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]
Dames & Moore - 20	5.3 [0.8] 7.2 [0.2]	5.5 [0.80] 7.2 [0.20]	5.5 [0.80] 7.2 [0.20]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]
Law - 126	4.6 [0.9] 4.9 [0.1]	5.5 [0.90] 5.7 [0.10]	5.5 [0.90] 5.7 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]
Rondout - 51	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	6.1 [0.30] 6.3 [0.55] 6.5 [0.15]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]
Weston - 107	5.4 [0.71] 6.0 [0.29]	6.6 [0.89] 7.2 [0.11]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 7.2 [0.10]
Woodward Clyde - B43	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	6.1 [0.10] 6.6 [0.40] 6.9 [0.40] 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% to 11%, 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 Mmax 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 Mmax updates are shown below in SER Figure 2.5.2-6.

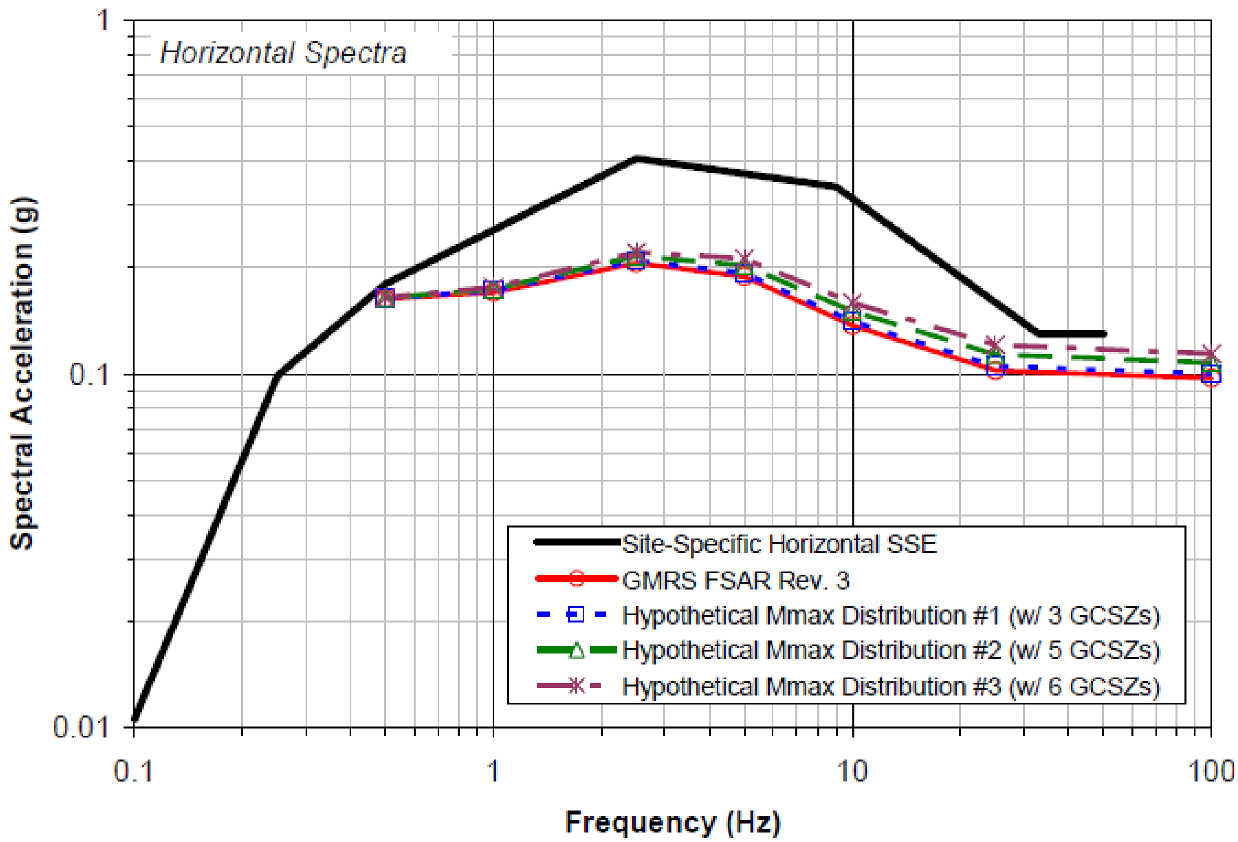

Figure 2.5.2-7, Response Spectra from the sensitivity test and comparison to the site-specific SSE

Based on the results of the applicant's sensitivity study, the staff concluded that the higher Mmax distribution originally proposed by the TI team does not significantly increase the GMRS for the STP site. Under either Scenarios 1 or 2, the increase in the GMRS is only about 10% at most. This result can be attributed to the significant epicentral distances of the two 2006 Gulf earthquakes to the STP site as well as the relatively sparse seismicity with 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 Mmax 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%. 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 Mmax update issue resolved. Therefore, RAI 02.05.02-13 and RAI 02.05.02-21 are resolved. 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.

Non-Gulf Seismic Sources

In addition to the Gulf of Mexico seismic sources, NRC 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. The applicant's response to **RAI 02.05.02-24** dated September 21, 2009 (ML092710096), updates FSAR Figure 2.S.2-8 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. The applicant's 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 concludes 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. The staff, therefore, considered this issue unresolved. Specifically, the Bechtel Group EST assigned a Pa of only 0.1 and Mmax 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 Mmax 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.

New Seismic Source Zones. 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, NRC 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 concludes 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.”

NRC 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. The applicant’s response to **RAI 02.05.02-14** dated September 4, 2008 (ML:082530449), states 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 provides 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 describes 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 an Mmax 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.

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, NRC 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. The applicant's response to **RAI 02.05.02-9** dated July 24, 2008 (ML082100162), points 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. The applicant's response to **RAI 02.05.02-20** dated July 20, 2009 (ML092030132), describes 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.

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. NRC 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. The applicant's response to **RAI 02.05.02-15** September 24, 2008 (ML082530449), states 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 notes 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. The applicant's response to **RAI 02.05.02-22** dated September 21, 2009 (ML092710096), refers 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 thus restates 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.

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, NRC 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.

2.5.2.4.3 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 presents 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 compares 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 concludes 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 concludes 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 bases 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, NRC 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 to September 2006 are depicted in SER Figure 2.5.2.6 as the red circles. The applicant's updated seismicity catalog is illustrated by the blue circles, which cover February 1985 to 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 2.5.2-6 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.

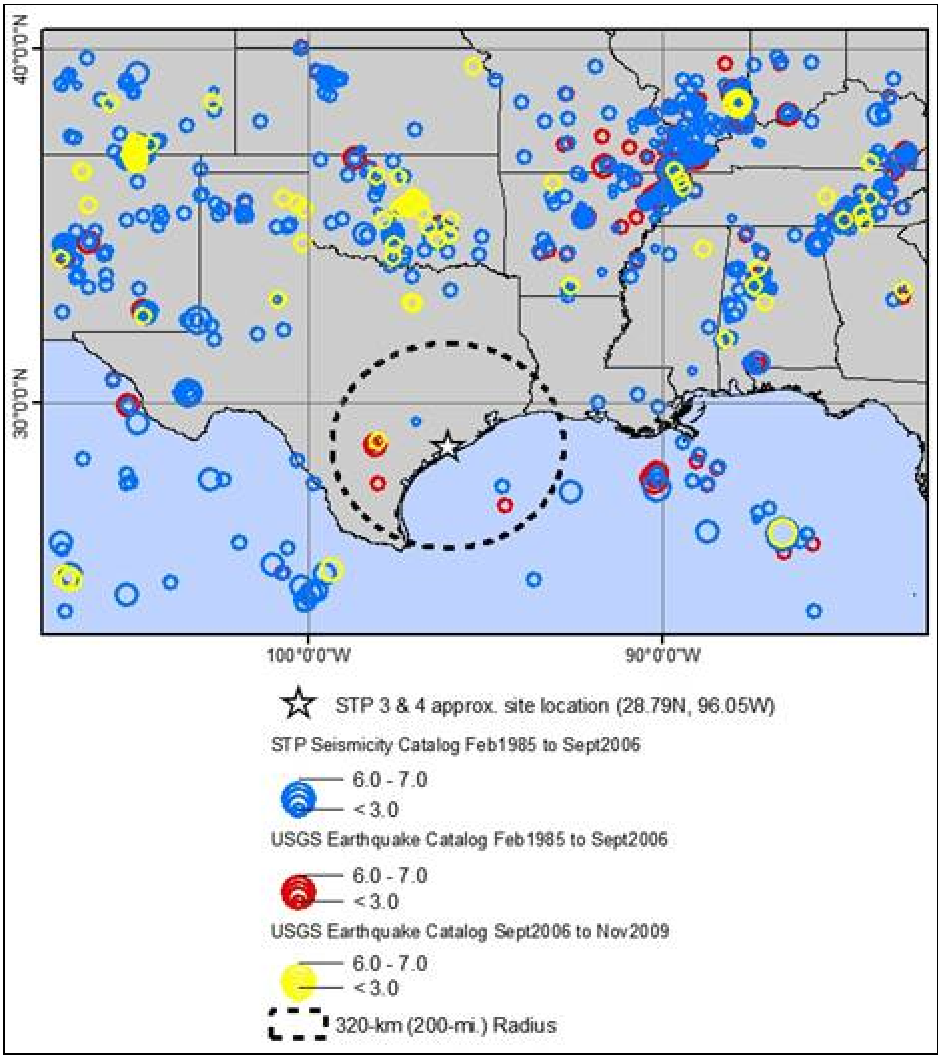

Figure 2.5.2-8. A Comparison of Events ($m_b \geq 3$) from the STP Units 3 and 4 Site Updated Earthquake Catalog from 1985 to 2006 (Blue Circles), the USGS Earthquake Catalog from 1985 to 2006 (Red Circles), and the USGS Earthquake Catalog from 2006 to 2009 (Yellow Circles). The Star Corresponds to the 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.

2.5.2.4.4 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 2.5.2.2.2 and 2.5.2.2.4 and the new ground motion models described in SER Subsection 2.5.2.2.4.

The NRC 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 2.5.2.4.2 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 states 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, 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 states 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, NRC 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.

The applicant's response to **RAI 02.05.02-11** dated August 12, 2008 (ML082270381) provides the software comparison results for all of the EPRI-SOG ESTs. The applicant notes significant differences in the validation results for all of the ESTs except Bechtel and Weston. The applicant states that the hazard calculations using its software are significantly lower than the original EPRI-SOG calculations for the following ESTs: Law (mean, 15th, 50th, and 85th fractile hazard); Rondout (15th fractile hazard); and Woodward-Clyde (15th 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 85th 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.

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 2.5.2-2 (reproduced from FSAR Table 2.5S.2-17) 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.

2.5.2.4.5 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 (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. NRC 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.

The applicant's response to **RAI 02.05.02-17** dated July 2, 2008 (ML081890239) included an FSAR Commitment (FSAR 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 to 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 FSAR 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.

In FSAR Subsection 2.5S.2.5, the applicant states 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, 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 FSAR COM 2.5S-1 (provided in the response to **RAI 02.05.02-17)**, the applicant indicates that "below 2,500 ft depth, a hard rock shear wave velocity of 2,830 m (9,285 ft/s) was assumed." The staff issued **RAI 02.05.02-25** requesting the applicant to clarify this discrepancy.

The applicant's response to **RAI 02.05.02-25** dated September 21, 2009 (ML092710096) indicates that the FSAR correctly states that an S-wave velocity of 2.8 km (9,200 ft/s) exists at a depth of more than 9,144 m (30,000 ft) below the ground surface. However, the applicant states 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 (9,200 ft/s). The applicant notes 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 FSAR 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 (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 commits to revise the FSAR Section 2.5S.4 to reflect the STP site-specific shear wave velocity profile. Verification of this FSAR commitment is being tracked as **Confirmatory Item 02.05.02-25.**

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 FSAR 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 (κ), 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 2.5.2.2.5, 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 assumes 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**, FSAR 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. The applicant's response to **RAI 02.05.02-26** dated September 21, 2009 (ML092710096) indicates 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 κ value calculated for the upper 182 m (600 ft) from the total or base case κ value (0.040 s) to obtain a residual κ value for the soil layers below 182 m (600 ft). The applicant then used the new S-wave velocity profile and the residual κ 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.

Site Response Methodology

In FSAR Subsection 2.5S.2.5.4, the applicant states that it used Random Vibration Technology (RVT) to calculate the site response. However, NRC 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. The applicant's response to **RAI 02.05.02-18** dated October 1, 2008 (ML082770138), describes using the Bechtel computer program SHAKE (P-SHAKE), which implements RVT, to calculate the site response. The applicant then summarizes the major steps in P-SHAKE and also provides a technical paper by Deng and Ostadan (2008), which describes 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 resolved.

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.

The applicant's response to **RAI 02.05.02-23** dated September 21, 2009 (ML092710096) indicates that the soil profile included in the attached paper (Deng and Ostadan, 2008) is a generic soil profile used for its numerical comparison of the P-SHAKE and the SHAKE programs. The applicant also commits 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. **RAI 02.05.02-23** is therefore resolved.

NRC Site Response Confirmatory Analysis

To determine the adequacy of the applicant's site-response calculations, NRC 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 and 2.5S.2-19 to represent the base rock input motions. The staff also used the strong motion duration values provided in FSAR Table 2.5S.2-20, 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.

2.5.2.4.6 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 2.5.2-4 and 2.5.2-5, respectively.

Because the applicant used the standard procedure outlined in RG 1.208 to develop both the horizontal and vertical GMRS, NRC staff concluded that the applicant's GMRS adequately represent the STP Units 3 and 4 site ground motion.

2.5.2.4.7 Staff Conclusions Regarding Vibratory Ground Motion

As set forth above, NRC staff reviewed the seismic information submitted by the applicant in STP Units 3 and 4 COL FSAR Section 2.5S.2 and all of the responses to the staff's RAIs. The staff found that the applicant has provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. In addition, 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 resolution of all RAIs has led the staff to the conclusion that 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. The staff concluded that the proposed COL site is acceptable from a geologic and seismologic standpoint and meets the requirements of 10 CFR 100.23.

This information addresses the STP Units 3 and 4 COL application and resolves COL License Information Item 2.24. In conclusion, the applicant provides sufficient information to satisfy 10 CFR 100.23.

2.5.2.5 *Post Combined License Activities*

There are no post COL activities related to this section.

2.5.2.6 *Conclusions*

NRC 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 with the exception of Confirmatory Item 02.05.02-25. As a result of this confirmatory item, the staff is unable to finalize the conclusions for this section relating to vibratory ground motion in accordance with NRC requirements.

REFERENCES

N.A. Abrahamson and J. Bommer, “Program on Technology Innovation: Truncation of the Lognormal Distribution and Value of the Standard Deviation for Ground Motion Models in the Central and Eastern United States,” Electric Power Research Institute (EPRI), Report 1014381, 2006.

American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” ASCE/SEI Report 43-05, 2005.

E. Collins, et al., “Quaternary Faulting in East Texas, University of Texas at Austin,” Bureau of Economic Geology, *Geological Circular,* 80:1, 1980.

Electric Power Research Institute (EPRI), “Probabilistic Seismic Hazard Evaluation at Nuclear Plant Sites in the Central and Eastern United States, Resolution of the Charleston Earthquake Issue,” EPRI Report 6395, 1989.

Electric Power Research Institute (EPRI), “Seismic Hazard Methodology for the Central and Eastern United States,” Volume 1, Part 2: Methodology (Revision 1), EPRI-SOG NP-4726, 1998.

Electric Power Research Institute (EPRI), “CEUS Ground Motion Project Final Report,” EPRI Technical Report 1009684, 2004.

Exelon Generation Company (EGC), “Early Site Permit ESP Application for the EGC ESP Site,” Revision 4, 2006.

A. Frankel, et al., “National Seismic-Hazard Maps: Documentation,” U.S. Geological Survey Open-File Report 96-532, 1996.

A. Frankel, et al., “Documentation for the 2002 Update of the National Seismic Hazard Maps,” U.S. Geological Survey, Open-File Report 02-420, 2002.

Frohlich and Davis, “Texas Earthquakes,” 240–255, 2002.

I.M. Idriss and J.I. Sun, “SHAKE91: A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits,” Department of Civil and Environmental Engineering, Center for Geotechnical Modeling, University of California, 1992.

W. Rinehart, et al., “Seismicity of Middle America: National Geophysical Data Center and National Earthquake Information Service,” 1982.

Schnabel et al., “Shake–A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites,” Earthquake Engineering Research Center (EERC) Report No. 72-12, 1972.

C.W. Stover and J.L. Coffman, “Seismicity of the United States, 1568–1989 (Revised),” United States Geological Survey Professional Paper 1527, 1993.

C.W Stover, G. Reagor, and S.T. Algermissen, “United States Earthquake Data File: United States Geological Survey, Open-File Report 84-225,” 1984

U.S. Nuclear Regulatory Commission, “Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-Consistent Ground Motion Spectra Guidelines,” R.K. McGuire, W.J. Silva, and C.J. Constantino, NUREG CR-6728, 2001.

U.S. Nuclear Regulatory Commission, “A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion,” Regulatory Guide (RG) 1.208, 2007

2.5.3 Surface Faulting

2.5.3.1 *Introduction*

FSAR Section 2.5S.3 of the STP Units 3 and 4 COL application is concerned with 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.

2.5.3.2 *Summary of Application*

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in 10 CFR Part 52, Appendix A. The applicant identifies no departures from the certified design and provides information to address COL License Information Item 2.25.

<u>COL License Information Item</u>

- STP 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 concludes 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 concludes 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 Units 3 and 4 footprint. The applicant applies 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.

2.5.3.2.1 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 states 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 1 and 2. Based on this information, the applicant concludes that there is no evidence for Quaternary age faulting in the STP site area (within an 8-km (5-mile) radius of the site).

2.5.3.2.2 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 concludes that there are no mapped faults in the STP site area that originate or extend into the crystalline basement rock. The applicant describes 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 discusses these previous and recent investigations in more detail in FSAR Subsection 2.5S.1.2.4.2. The applicant describes the results of recent investigations with respect to growth fault "I" (Matagorda STP 12I) that were previously documented in the UFSAR for 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 states 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 concludes 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 concludes 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.

2.5.3.2.3 Correlation of Earthquakes with Capable Tectonic Sources

In FSAR Subsection 2.5S.3.3, the applicant concludes 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.

2.5.3.2.4 Ages of Most Recent Deformations

In FSAR Subsection 2.5S.3.4, the applicant concludes 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 describes the results from previous growth fault investigations and concludes 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.

2.5.3.2.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Sources

In FSAR Subsection 2.5S.3.5, the applicant concludes 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.

2.5.3.2.6 Characterization of Capable Tectonic Sources

In FSAR Subsection 2.5S.3.6, the applicant concludes that no capable tectonic structures exist within the STP site area.

2.5.3.2.7 Designation of Zones of Quaternary Deformation in the Site Region

In FSAR Subsection 2.5S.3.7, the applicant states that no zones of Quaternary tectonic deformation exist in the site area. The applicant notes 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.

2.5.3.2.8 Potential for Surface Tectonic Deformation at the Site

In FSAR Subsection 2.5S.3.8, the applicant states 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 discusses 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 concludes 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 states that subsidence due to dewatering could reach maximum levels of 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.

2.5.3.3 *Regulatory Basis*

The regulatory basis and acceptance criteria for reviewing COL License Information Item 2.25 are in Section 2.5.1 of NUREG–0800. 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 RG 1.208, RG 1.132, RG 1.198, RG 4.7, and RG 1.206.

2.5.3.4 *Technical Evaluation*

NRC staff reviewed the information in Section 2.5S.3 of the STP Units 3 and 4 COL FSAR:

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

- STP COL License Information Item 2.25 Surface Faulting

NRC staff reviewed the applicant's information in FSAR Section 2.5S.3 that addresses this item. Specific information from the applicant 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 concludes 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 concludes 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.

2.5.3.4.1 Geologic, Seismic and Geophysical Investigations for Surface Deformation

NRC 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. The applicant's response to **RAI 02.05.03-1** dated August 27, 2008 (ML082490086), states 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.

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.

2.5.3.4.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

The NRC 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 concludes 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.

2.5.3.4.3 Correlation of Earthquakes with Capable Tectonic Sources

NRC staff reviewed FSAR Subsection 2.5S.3.3, including the applicant's evaluation of seismicity data for the STP site. The applicant concludes 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. The applicant's response to **RAI 02.05.03-3** dated September 4, 2008 (ML0825530449), acknowledges 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 Gulf Coast Seismic Zone (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 states 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 concludes 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 includes 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 2.5.S.2.4.2 of this SER. **RAI 02.05.03-3** is resolved.

The staff issued **RAI 02.05.03-2** requesting the applicant to discuss any potential effects of migrating seismicity at the STP site. The applicant's response to **RAI 02.05.03-2** dated September 4, 2008 (ML082530449), explains that in areas like the New Madrid Seismic Zone (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 states 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.

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.

2.5.3.4.4 Ages of Most Recent Deformation

NRC 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.

2.5.3.4.5 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 concludes that growth faults are not considered to be tectonic structures and are not linked to any capable regional tectonic sources.

NRC 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.

2.5.3.4.6 Characterization of Capable Tectonic Sources

NRC 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 NRC staff and USGS advisors, and (4) a lack of identified tectonic structures in the existing literature for the STP area.

2.5.3.4.7 Designation of Zones of Quaternary Deformation at the Site

NRC staff reviewed the applicant's discussion of Quaternary zones of deformation in FSAR Subsection 2.5S.3.7. The applicant concludes 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.

2.5.3.4.8 Potential for Surface Tectonic Deformation at the Site

The applicant concludes in FSAR Subsection 2.5S.3.8 that the potential for tectonic and non-tectonic surface deformation at the site is negligible. NRC 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 2.5.3.4.2 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. The applicant's response to **RAI 02.05.03-4** dated June 26, 2008 (ML081970231) includes the necessary corrections and also updates the references in the latest revision of the FSAR. Therefore, **RAI 02.05.03-4** is resolved.

2.5.3.5 *Post Combined License Activities*

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

2.5.3.6 *Conclusion*

NRC 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 concluded 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 concluded 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.

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 *Introduction*

STP COL FSAR Section 2.5.4 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 provided by the applicant in STP COL FSAR Section 2.5.4 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.

2.5.4.2 *Summary of Application*

The applicant incorporates by reference Section 2.1 of the certified ABWR DCD referenced in Appendix A to 10 CFR Part 52, with no departures. The applicant provides supplemental information to address COL License Information Items 2.26, 2.27, 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 provides 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 provides 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 provides 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 provides 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 provides 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 provides 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 provides 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 provides 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 provides 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) 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 provides 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 provides 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 provides 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 provides 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.”

2.5.4.2.1 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.

2.5.4.2.2 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 states 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, 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 2.5.4-1 (FSAR Table 2.5S.4-16) 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 notes 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 plans 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 found 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 notes 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 found 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 concludes

that special protection may be required if metals are to be placed against the soils. The applicant also notes 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 notes 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 states 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.

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 states 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 2.5.4-2 presents the type and number of tests performed and the standards followed.

Table 2.5.4-1. Summary of Average Geotechnical Engineering Parameters (FSAR Table 2.5S.4-16)

	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)
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)	-

	A	B	C	D	E	F	H	J CLAY	J SAND	K CLAY	K SAND/ SILT	L	M	N CLAY	N SAND
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) (μ_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	-	-	-	-	-	-	-	-
-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	-	-	-	-	-	-	-	-

	A	B	C	D	E	F	H	J CLAY	J SAND	K CLAY	K SAND/ SILT	L	M	N CLAY	N SAND
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	-

Table 2.5.4-2. Field Testing Summary (FSAR Table 2.5S.4-1)

FIELD TEST	INDUSTRY STANDARD	NUMBER OF TESTS
Borings (B)	ASTM D 1586 (1999) ASTM D 1587 (2000)	132
SPT Hammer Energy Measurements	ASTM D 6066 (2004) 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) IEEE 81 (1983)	4
Suspension P-S Velocity Logging	ASCE Ohya (1986)	10

<u>Laboratory Testing Program</u>. 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.

<u>Subsurface Investigation (STP Units 3 and 4).</u> 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.

<u>Boring and Sampling</u>. 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 2.5.4-3 (FSAR Table 2.5S.4-7) identifies the type, number, and industry standard followed for each type of laboratory test.

Table 2.5.4-3. 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) ASTM D 6913 (2004)	257
Specific Gravity	ASTM D 854 (2006)	107
Unit Weight	ASTM Standards	141
Unconsolidated Undrained Triaxial Strength	ASTM D 2850 (2003)	76
Unconfined Compressive Strength	ASTM D 2166 (2006)	25

LABORATORY TEST	INDUSTRY STANDARD	NUMBER OF TESTS
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

2.5.4.2.3 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.

2.5.4.2.4 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 2.5.4-4 (FSAR Section 2.5S.4.4.2.1) presents the minimum, maximum, and average shear wave velocity values in various soil strata from STP Units 3 and 4.

Table 2.5.4-4. Minimum, Maximum, and Average 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 states that the maximum depth tested by the seismic CPTs was approximately 28.9 m (95 ft) below the ground surface. SER Table 2.5.4-5 (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 2.5.4-5. 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 2.5.4-1 (FSAR Figure 2.5S.4-45) 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 notes that the shear wave velocity increases from about 304 m/s (1,000 fps) to about 457 m/s (1,500 fps).

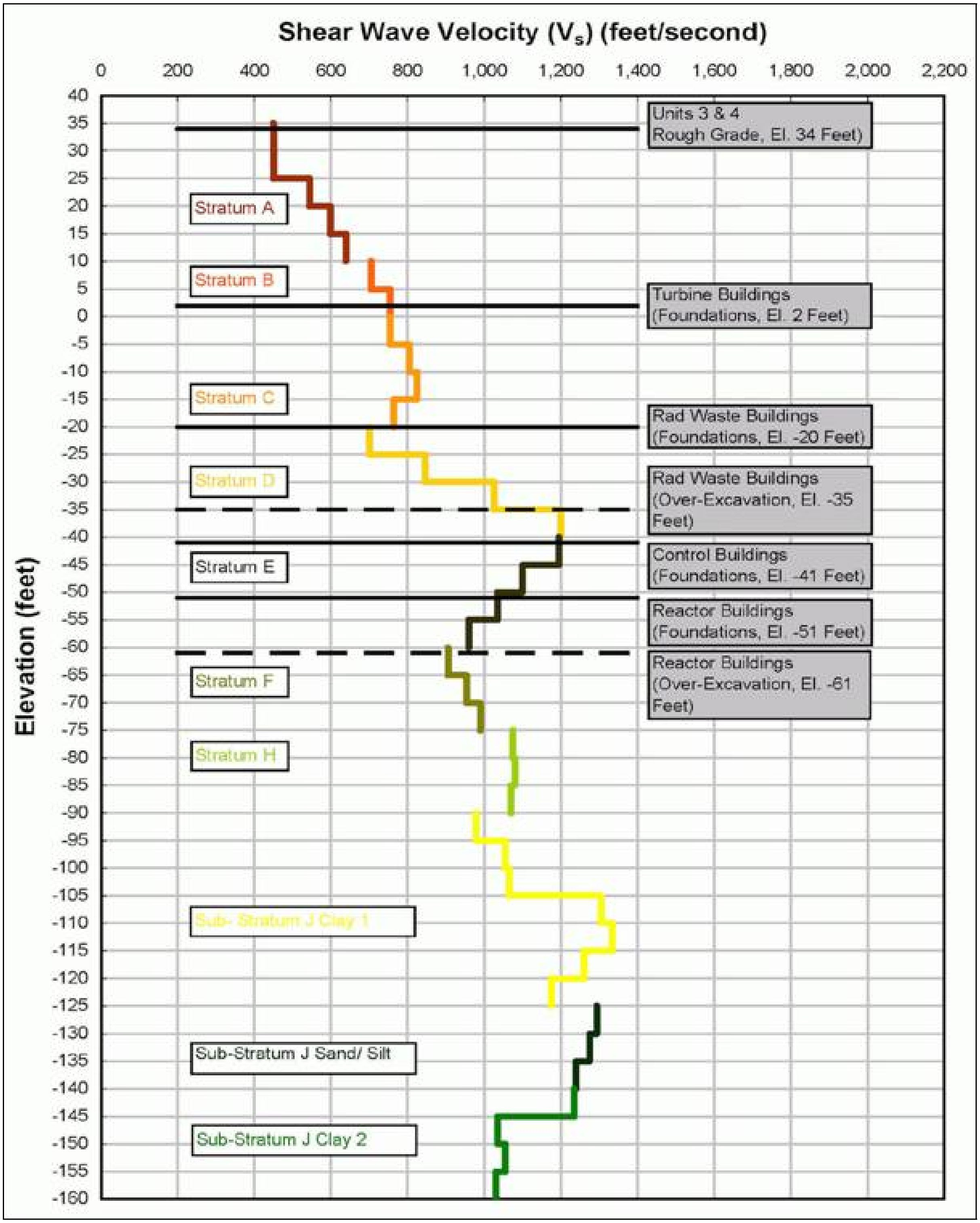

Figure 2.5.4-1. Shear Wave Velocity Profile of Strata A to J (FSAR Figure 2.5S.4-45)

2.5.4.2.5 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 proposes using a combination of excavation slopes and temporary retaining structures to reach the foundation level. Finally, the applicant states 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 estimates 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 expects 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 applicant will found the reactor buildings for STP Units 3 and 4 on concrete fill but will found other major structures directly on dense sand strata or on structural fill. SER Figure 2.5.4-2 (FSAR Figure 2.5S.4-49A) 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 states 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 (FS) of 1.3 for the temporary excavation slopes.

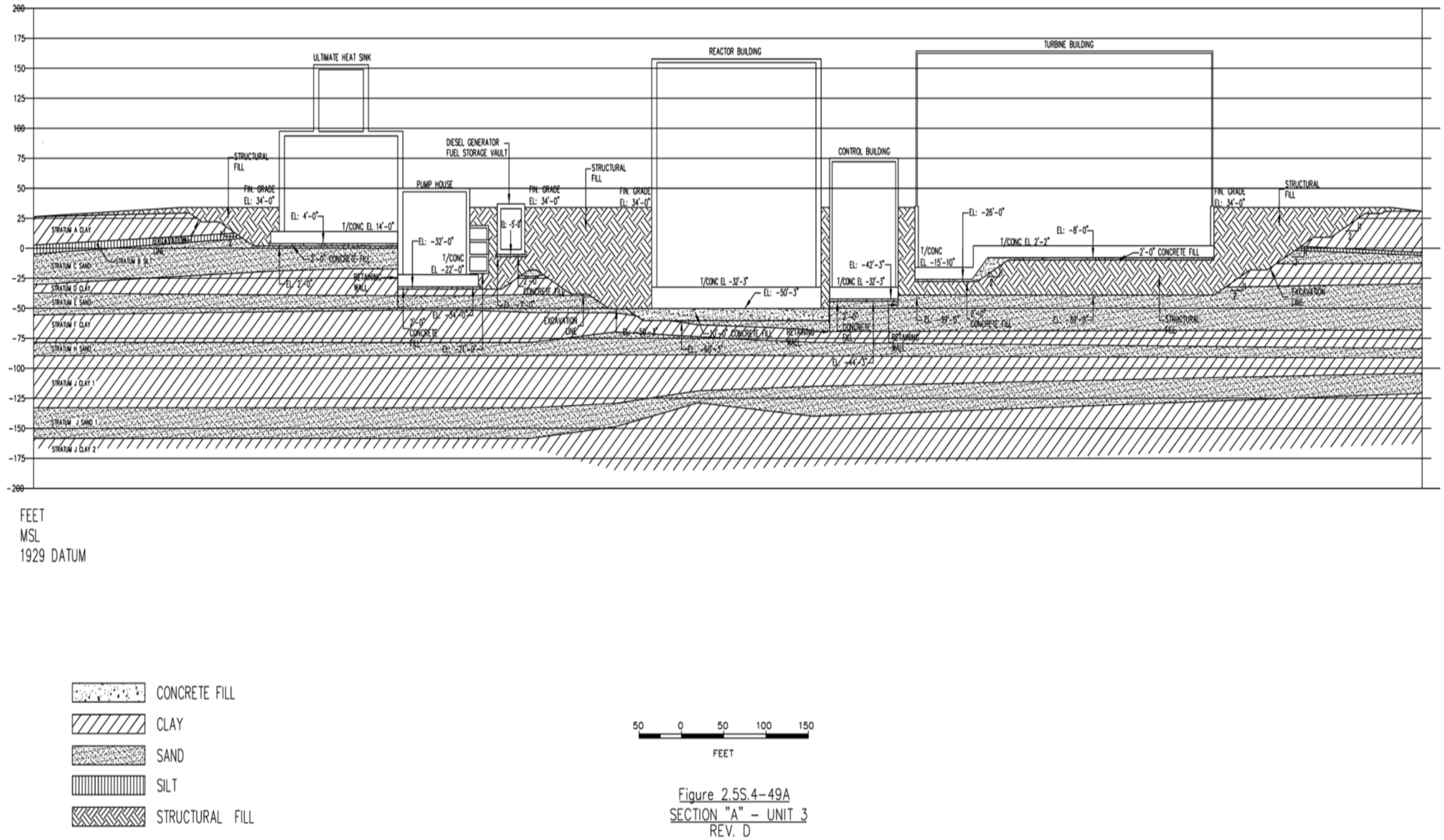

Figure 2.5.4-2. Section A Unit 3 (FSAR Figure 2.5S.4-49A)

Retaining Structures for Adjacent Foundations. The applicant states 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 describes 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 states 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 states 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 3 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 states 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.

2.5.4.2.6 Ground Water Conditions

In FSAR Subsection 2.5S.4.6, the applicant describes the ground water conditions at the STP Unit 3 and 4 sites. The applicant provides 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 describes the upper water table, which is at an El. of 5.2 m (26.5 ft), as a perched condition that will disappear with the excavation. The applicant selected the ground water level at an El. of 7.7 m (26.5 ft) for liquefaction analysis purposes.

Construction Stage Dewatering

The applicant states 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 notes that dewatering operations will cease and the ground water will return to normal levels.

Analysis and Interpretation of Seepage

The applicant states 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.

2.5.4.2.7 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 states 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 refers 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 2.5.4-1 (FSAR Figure 2.5S.4-45) shows the average shear wave velocity profile for the upper 49 m (160 ft).

The applicant summarizes 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 concludes 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 concludes that the results are relatively consistent within variations of about ±30.48 m/s (±100 fps). The applicant notes one exception between elevations of approximately -12 to -32 m (-40 to -105 ft), but also notes 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 notes 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 estimates that the top depth of pre-Cretaceous bedrock occurs at approximately 10,515 m (34,500 ft) below the ground surface. The applicant states 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 notes 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. In COM 2.5S-1, the applicant commits 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 2.5.4-3 (FSAR Figure 2.5S.4-58) 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 2.5.4-4 (FSAR Figure 2.5S.4-60) depicts the applicant’s curves for the cohesionless soil strata. The applicant limited damping to 15 percent.

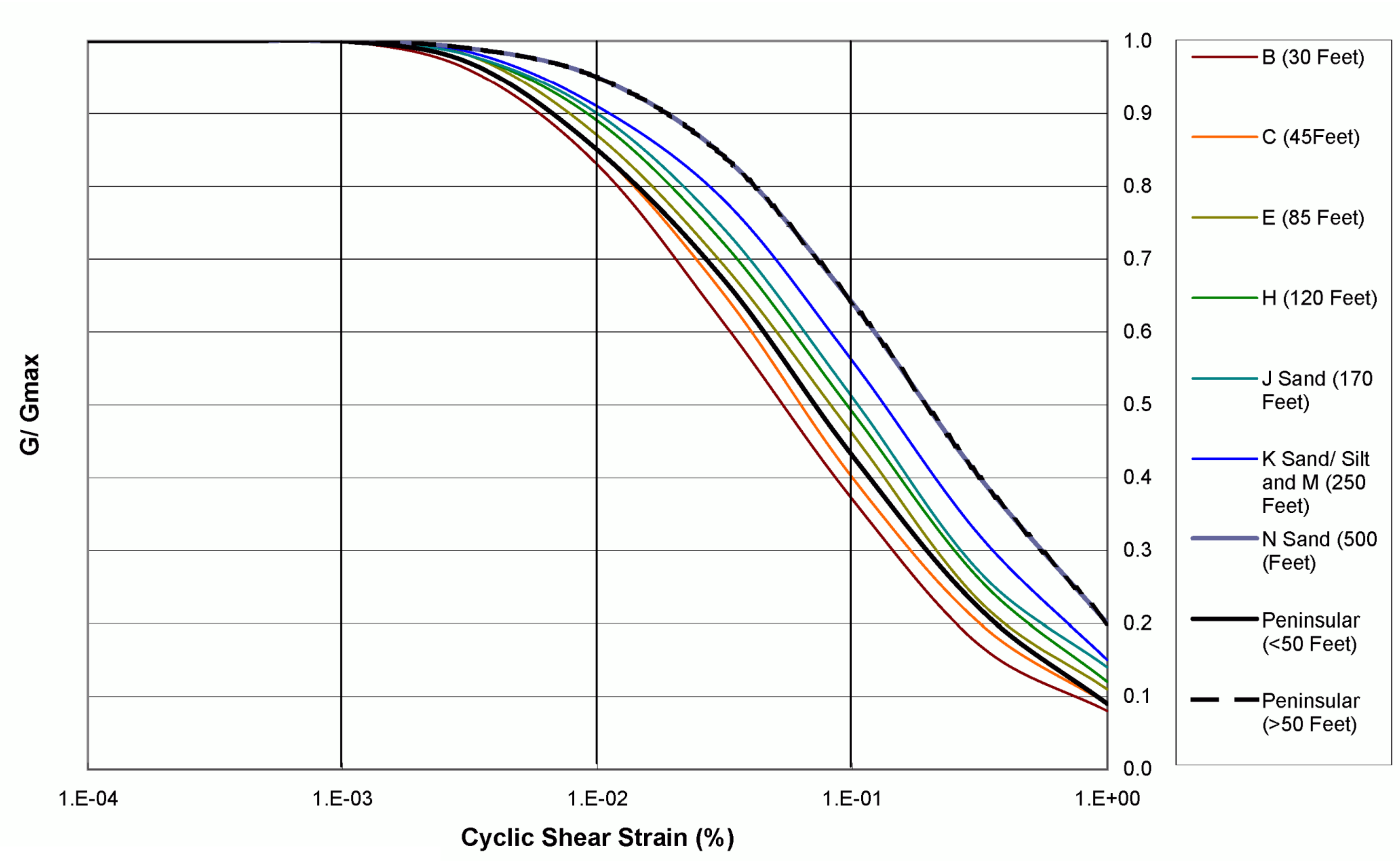

Figure 2.5.4-3. Selected Shear Modulus Degradation Curve for Cohesionless Soils (FSAR Figure 2.5S.4-58)

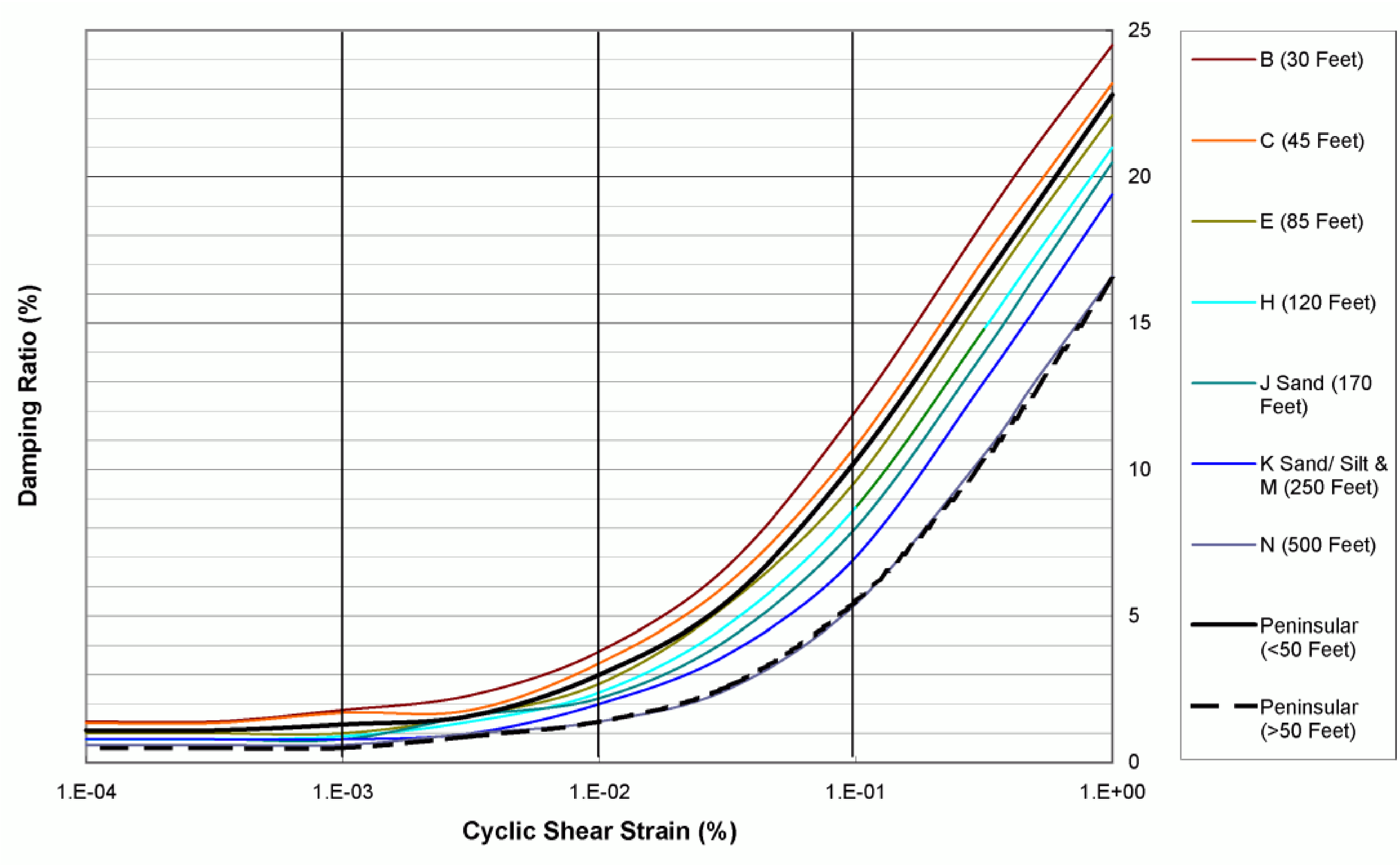

Figure 2.5.4-4. Selected Damping Ratio Curve for Cohesionless Soils (FSAR Figure 2.5S.4-60)

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 (2004) curves. The applicant compared the curves developed from the RCTS results to the EPRI (2004) curves and concluded that there was good agreement. However, because the applicant initially performed a limited number of RCTS tests, COM 2.5S-1 commits the applicant to modify the dynamic soil model if warranted, by further site-specific RCTS test results. To that end, 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 2.5.4-3.

With regard to the rock, the applicant states 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.

2.5.4.2.8 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 (Mw) 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 notes 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.

Liquefaction Evaluation Methodology. 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 notes 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 shear wave velocity (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 notes 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 states that approximately 71.6 percent of the 1,687 tests performed demonstrated Vs1≥Vs1*, implying that most of the site soils are too dense to liquefy. Based on these results, the applicant concludes 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.

2.5.4.2.9 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.

2.5.4.2.10 Static Stability

STP Units 1 and 2 Foundations

In FSAR Subsection 2.5S.4.10.1, the applicant describes the previous experience with STP Units 1 and 2. The applicant states 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 describes the subsurface conditions and soil properties used to analyze safety-related Seismic Category I foundations. FSAR Table 2.5S.4-16 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 2.5.4-5 (FSAR Figure 2.5S.4-71) 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 states 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 states 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 describes the pseudo-elastic method and the one-dimensional consolidation method of analysis used to estimate settlement. The applicant states that the pseudo-elastic approach is suitable for granular deposits and clay strata because the clay strata are overconsolidated. The applicant notes that for the most part, the preconsolidation pressures are not exceeded.

However, in those instances where the applied stresses exceeded the preconsolidation pressure, the applicant used consolidation theory to compute the settlements. The applicant also states 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 summarizes the estimated settlement results calculated at the center, corner, and middle edges of the foundation mats. The applicant states 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 notes that settlements were based on the assumption of a flexible mat, which produces overly conservative results compared to a rigid mat. The applicant introduces a correction methodology to estimate rigid foundation settlements from flexible foundation settlements based on design guidance found in Bowles (1997). The applicant states 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 notes 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 notes 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 states 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 2.5.4-6 presents the estimated foundation settlements for key structures at STP Units 3 and 4.

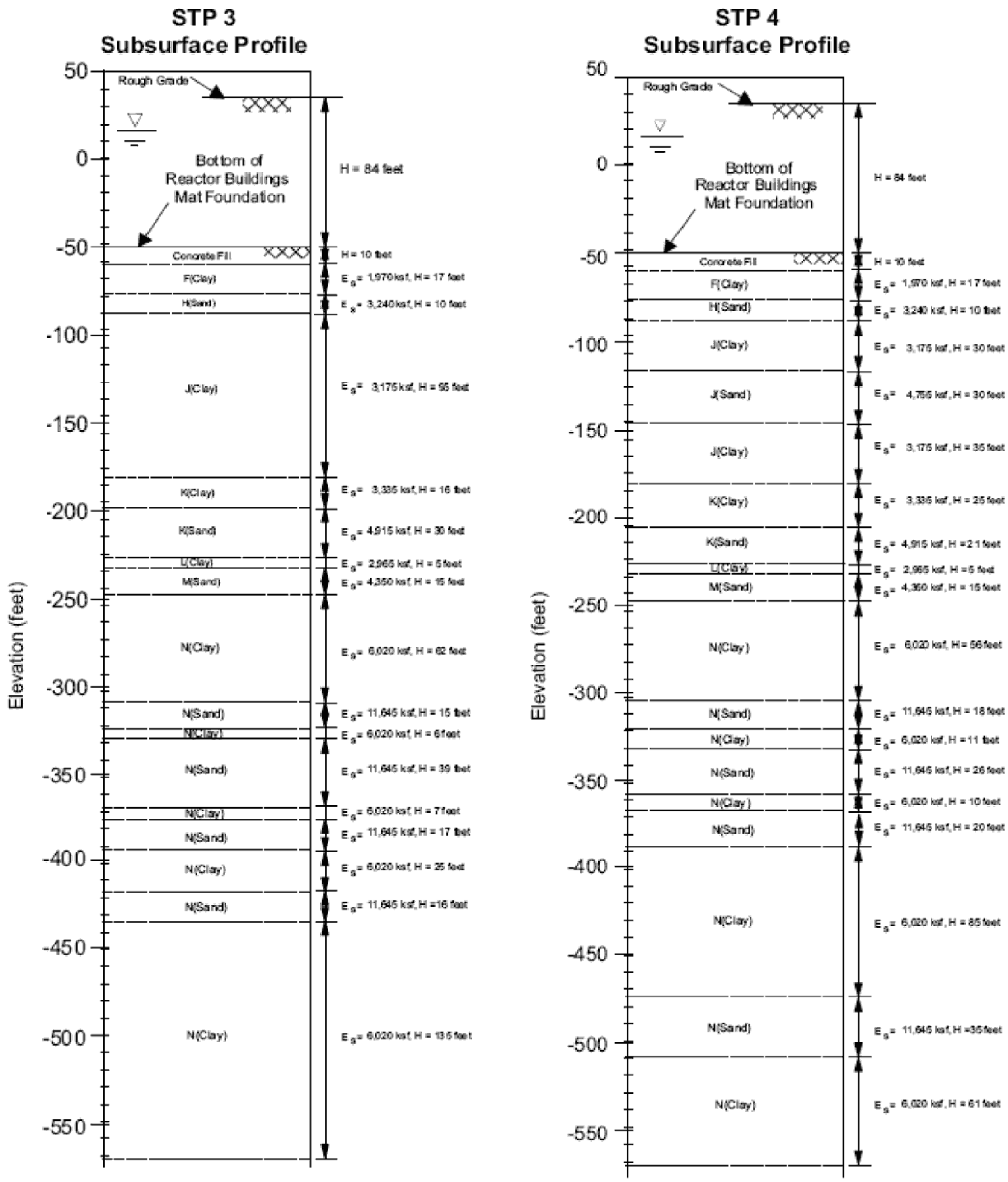

Figure 2.5.4-5. Adopted Subsurface Profiles for STP Units 3 and 4 Reactor Buildings (FSAR Figure 2.5S.4-71)

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 states 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.

Table 2.5.4-6. 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
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

Sample Earth Pressure Diagram. FSAR Figures 2.5S.4-76 and 2.5S.4-77 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)
- (γ) = 1,922 kg/m^3 (120 pcf) (Backfill)
- PGA = 0.10 g
- Uniform Surcharge = 23.9 kPa (500 psf)

Until the actual backfill properties and surcharge loads are known, the applicant presents the active and at-rest pressure diagrams for illustration purposes only. SER Figure 2.5.4-6 (FSAR Figure 2.5S.4-76) illustrates a sample diagram showing the active lateral earth pressures.

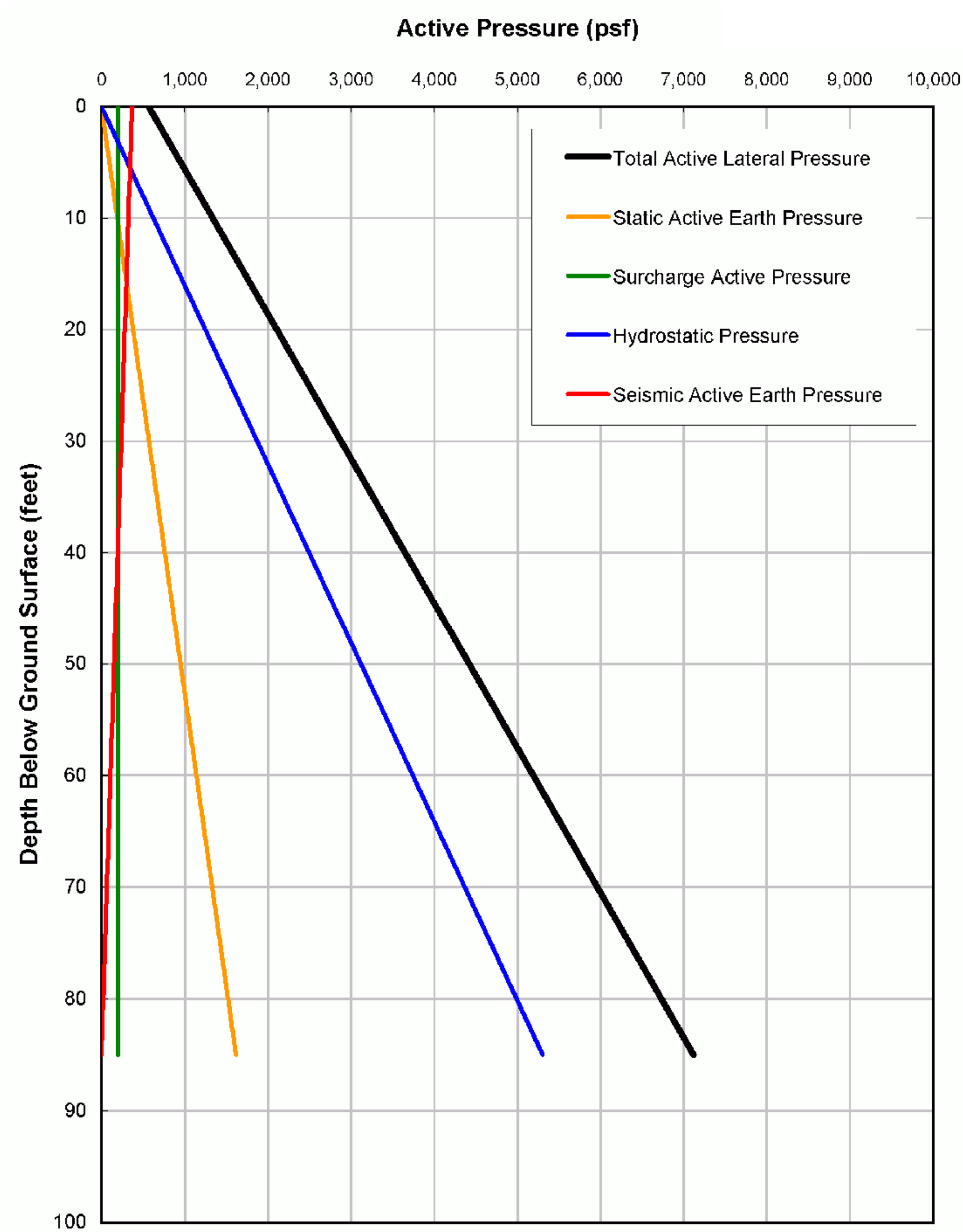

Figure 2.5.4-6. Sample Active Lateral Earth Pressure Diagram (FSAR Figure 2.5S.4-76)

2.5.4.2.11 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.

2.5.4.2.12 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 describes 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.

2.5.4.3 *Regulatory Basis*

The regulatory basis and acceptance criteria for reviewing COL License Information Items 2.26, 2.27, 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. The applicable regulatory requirements for reviewing geologic and seismic information are as follows:

(1) 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.

(2) 10 CFR Part 50, Appendix A, General Design Criterion 1 (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.

(3) 10 CFR Part 50, Appendix A, General Design Criterion 2 (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.

(4) 10 CFR Part 50, Appendix A, General Design Criterion 44 (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.

(5) 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.

(6) 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.

(7) 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.

(8) 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:

(1) 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.

(2) 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.

(3) 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.

(4) 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.

(5) 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.

(6) 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.

(7) 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.

(8) 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.

(9) 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.

(10) 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.

(11) 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, "Site Investigations for Foundations of Nuclear Power Plants"; RG 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants"; RG 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites"; and RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)."

2.5.4.4 *Technical Evaluation*

NRC staff reviewed the information in Section 2.5S.4 of the STP Units 3 and 4 COL FSAR:

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

- 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

NRC staff evaluated the above list of COL license information items in the following subsections.

2.5.4.4.1 Description of Site Geologic Features

FSAR Subsection 2.5.4.1 refers to FSAR Subsections 2.5.1.1 and 2.5.1.2 for detailed descriptions of the regional geology and site geology, respectively. SER Subsections 2.5.1.4.1 and 2.5.1.4.2 provide the technical evaluation of the regional and site geologic features.

2.5.4.4.2 Properties of Subsurface Materials

SER Subsection 2.5.4.2.2 summarizes FSAR Subsection 2.5.4.2. The applicant performed an exploratory program that included SPTs, CPTs, undisturbed sampling, test pits, field testing, geophysical surveys, and laboratory testing. The applicant states 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. NRC 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. NRC 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.

The response to **RAI 02.05.04-6** dated July 9, 2008 (ML081960070), demonstrates that the applicant 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 concludes 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 notes 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.

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.

The applicant's response to **RAI 02.05.04-22** dated July 20, 2009 (ML092030132), describes encountered fissures, slickensides, and calcareous deposits in the samples taken at the site that are consistent with the soils documented in Mahar and O'Neil (1983). The applicant explains 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 notes 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 indicates 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 reasons 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. Accordingly, **RAI 02.05.04-22** is resolved.

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. NRC 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 and 2.5S.4-30 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. 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.

The applicant's response to **RAI 02.5.04-7** dated July 9, 2008 (ML081960070), notes 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 concludes that the average values are representative of the OCR values derived from the CPT results. The applicant adds that there is no geologic mechanism for an OCR of less than 1 in Pleistocene-age samples. The applicant also addresses 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 3 and 2.5 for STP 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 states 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. **RAI 02.05.04-7** is therefore resolved.

FSAR Figure 2.5S.4-28 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 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.

The applicant's response to **RAI 02.05.04-8** dated July 9, 2008 (ML081960070), states that in most soil profiles, the OCR decreases with increasing depth and eventually reaches unity, or very close to unity. The applicant also explains 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 states 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 notes 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.

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. NRC 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 tsf), the corrected N-value would be unconservative.

The applicant's response to **RAI 02.05.04-9** dated August 12, 2008 (ML082270381), states 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 (1 tsf) 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 states 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 concludes 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 notes that the S_u value selected for design is less than the S_u value based on the N-value. The applicant concludes 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. Accordingly, **RAI 02.05.04-9** is therefore resolved.

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.

The applicant's response to **RAI 02.05.04-10** dated August 12, 2008 (ML082270381), states 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 concludes 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 concludes 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 concludes 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.

In FSAR Subsection 2.5S.4.2.1.6, the applicant determined the drained strength parameters for Stratum F, but then assumed an effective phi (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.

The applicant's response to **RAI 02.05.04-11** dated August 12, 2008, states that there was an error in reporting the cohesion of Stratum F, Stratum D, and Stratum J Clay. This error was corrected in Rev. 3 of the COL application.. Additionally, the applicant explains 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 Manual DM 7.02.

The staff reviewed the referenced table in the Naval Facilities Command Engineering 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. Therefore, **RAI 02.05.04-11** is resolved.

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. NRC 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.

The applicant's response to **RAI 02.05.04-17** dated April 1, 2009 (ML090930717), proposes 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. 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. Accordingly, **RAI 02.05.04-17** is resolved.

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, the applicant's response to **RAI 02.05.04-18** dated April 1, 2009, restates 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 states 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. Accordingly, **RAI 02.05.04-18** is resolved.

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 02.05.04-19** 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.

The applicant's response to **RAI 02.05.04-19** dated April 1, 2009, includes changes to the FSAR in conjunction with the **RAI 02.05.04-13**, Supplement 1 response. The applicant uses the elastic modulus of the various soil strata to estimate settlements because the soils behave as overconsolidated. Furthermore, the applicant bases 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 indicates 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 states 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 concludes 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.

In FSAR Section 2.5.4.2, the applicant calculates 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.

The applicant's response to **RAI 02.05.04-20** dated April 1, 2009, proposes changes in the FSAR mark-up submitted in conjunction with the **RAI 02.05.04-13**, Supplement 1 response. The applicant describes 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 notes 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 concludes 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.

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.

The applicant's response to **RAI 02.05.04-23** dated July 20, 2009 (ML092030132), states 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 notes 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 uses 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 confirms 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 concludes 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. Accordingly, **RAI 02.05.04-23** is resolved.

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.

The response to **RAI 02.05.04-25** dated August 10, 2009 (ML092250658), notes 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 justifies 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. Accordingly, **RAI 02.05.04-25** is resolved.

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.

The applicant's response to **RAI 02.05.04-26** dated August 10, 2009, states 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 concludes 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. Accordingly, the **RAI 02.05.04-26** is resolved.

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 describes 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 uses the results of these investigations, as presented in SER Table 2.5.4-1 (FSAR Table 2.5S.4-16) 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

NRC 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.

2.5.4.4.3 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 NRC 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, 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.

The applicant's response to **RAI 02.05.04-1** dated October 21, 2008 (ML082970562), provides 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 includes 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 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 Unit 4 power block areas. Accordingly, **RAI 02.05.04-1** is resolved.

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.

2.5.4.4.4 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). NRC 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.

The applicant's response to **RAI 02.05.04-12** dated September 10, 2008 (ML082560248), states that since the measured shear wave velocities do not meet the minimum value of 304 m/sec (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.

2.5.4.4.5 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. NRC 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.

The applicant's response to **RAI 02.05.04-2** dated October 1, 2008 (ML082770138), describes 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 states 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 describes 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 identifies 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 states 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 presents 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 notes that the crane loading will minimally affect loads on the reinforced concrete wall.

The applicant performed slope stability analyses using the STABL7 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 states 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 STABL7 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.

The applicant's response to **RAI 02.05.04-24** dated August 10, 2009 (ML092250658), refers 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 considers 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 concludes 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. Accordingly, **RAI 02.05.04-24** is resolved. The resolution of **RAI 02.05.04-24** also resolves **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. NRC 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.

The applicant’s response to **RAI 02.05.04-4** dated October 1, 2008 (ML082770138), provides 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 includes 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. Accordingly, **RAI 02.05.04-4** is resolved.

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 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. NRC 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.

The applicant’s response to **RAI 02.05.04-27** dated August 10, 2009 (ML092250658), and **RAI 02.05.04-31** dated October 21, 2009 (ML092890084), refers to the response submitted in response to **RAI 14.03.02-6**, where the applicant states plans to use quality control procedures to verify key parameters of the backfill materials in FSAR Table 2.5S.4.5.3-1. 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 states 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 establishes 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.

The applicant's response to **RAI 02.05.04-33** dated January 21, 2010 (ML100250137), refers 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. Finally, the applicant plans to revise FSAR Subsection 2.5S.4.5.3 and Table 2.5S.4.5.3-1 to include this information.

In the supplemental response to **RAI 02.05.04-33** dated March 15, 2010 (ML100770389), the applicant proposes to incorporate the commitments in the initial response to **RAI 02.05.04-33** into an ITAAC. The applicant also revises the compaction specification to be consistent with the Quality Assurance Program requirements. In addition to proposing a new ITAAC, the applicant also proposes revising the FSAR to include more detailed compaction specifications. NRC staff reviewed the proposed revisions to the FSAR. Verification of the FSAR revisions, particularly Subsection 2.5S.4.5.3 and Table 2.5S.4.5.3-1, as proposed by the applicant in its responses to RAI 02.05.04-33 is being tracked as **Confirmatory Item 02.05.04-33**.

The staff also 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 also 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.

The applicant's response to **RAI 02.05.04-34** dated April 1, 2010 (ML100980067), addresses 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 test 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 proposes 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 updates FSAR Subsection 2.5S.4.7.3.7 to refer to the revised backfill ITAAC.

The staff reviewed the proposed revisions to the FSAR. Verification of the proposed FSAR updates, including Subsections 2.5S.4.5.3 and 2.5S.4.7.3.7, is being tracked as **Confirmatory Item 02.05.04-34**.

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 the applicant update the proposed ITAACs to reflect demonstrations that the assumption in the safety analyses are verified and consistent with the requirements of 10 CFR 100.23.

The applicant's response to **RAI 02.05.04-36** dated June 3, 2010 (ML101590397), addresses each ITAAC, backfill properties, shear wave velocity, engineering properties of backfill, and settlement, separately as shown in Table 2.5.4-7 (COL application Part 9, Section 3, Table 3.0-11) and Table 2.5.4-10 (COL application Part 9, Section 3, Table 3.0-13). The staff's evaluation of the settlement ITAAC is presented in Section 2.5.4.4.10 of this SER.

Table 2.5.4-7 Backfill Under Seismic Category I Structures (COL Application Part9, 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.
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 proposes 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 states 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. NRC 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, and 02.05.04-34**.

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 response has not been submitted on the docket and is **Open Item 02.05.04-37**.

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.

The applicant's response to **RAI 02.05.04-32** dated December 21, 2009 (ML093580191), provides 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. Accordingly, **RAI 02.05.04-32** is resolved.

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 describes excavation and foundation monitoring plans that sufficiently address the COL license information items. However, the applicant does not discuss the backfill criteria and specifications for NRC staff to adequately evaluate. In response to supplemental RAI 02.05.04-33 and subsequent supplemental RAIs 02.05.04-34 and 02.05.04-36, the applicant provide 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 was 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

NRC 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, supplemental RAIs 02.05.04-34 and 02.05.04-36, 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. The staff identified **Confirmatory Items 02.05.04-33 and 02.05.04-34** to ensure that the responses to the respective RAIs have been adequately incorporated into the revised FSAR. The staff also developed **Open Item 2.5.4-37**, which requests the applicant to provide the Backfill ITAAC tests to be performed, as well as the testing frequency that will be followed in the ITAAC. As a result of the above **confirmatory items and open item**, the staff was unable to finalize the conclusions relating to the "Excavation and Backfill", in accordance with NRC requirements.

2.5.4.4.6 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. NRC staff issued **RAI 02.05.04-3** requesting the applicant to provide the dewatering plan and the dewatering monitoring plan.

The applicant's response to **RAI 02.05.04-3** dated October 1, 2008 (ML82770138), provides 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 describes 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 0.914 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 9.14 m (30 ft) from the edge of the excavation and continuously around the perimeter of the excavation. The applicant notes that the minimum permeability of the slurry wall will be 1×10^{-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 provides 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 0.914 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. Accordingly, **RAI 02.05.04-3** is resolved.

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.

The applicant's response to **RAI 02.05.04-16** dated April 1, 2009 (ML090930717), clarifies 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 states 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. Accordingly, **RAI 02.05.04-16** is resolved.

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.

2.5.4.4.7 Response of Soil and Rock to Dynamic Loading

NRC 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.

The applicant's response to **RAI 02.05.02-17** dated July 2, 2008 (ML081890239), revises 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 2.5.4-7 and 2.5.4-8 (FSAR Figures 2.5S.4-62 and 2.5S.4-66) 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 2.5.4-7. Similarly, the applicant selected generic shear modulus degradation curves for cohesive soil strata based on the PIs of the strata. SER Figures 2.5.4-9 and 2.5.4-10 (FSAR Figures 2.5S.4-64 and 2.5S.4-65) 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 concludes 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 refers 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, 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 2.5.4-1 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 2.5.4-11 (FSAR Figure 2.5S.2-47) 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.

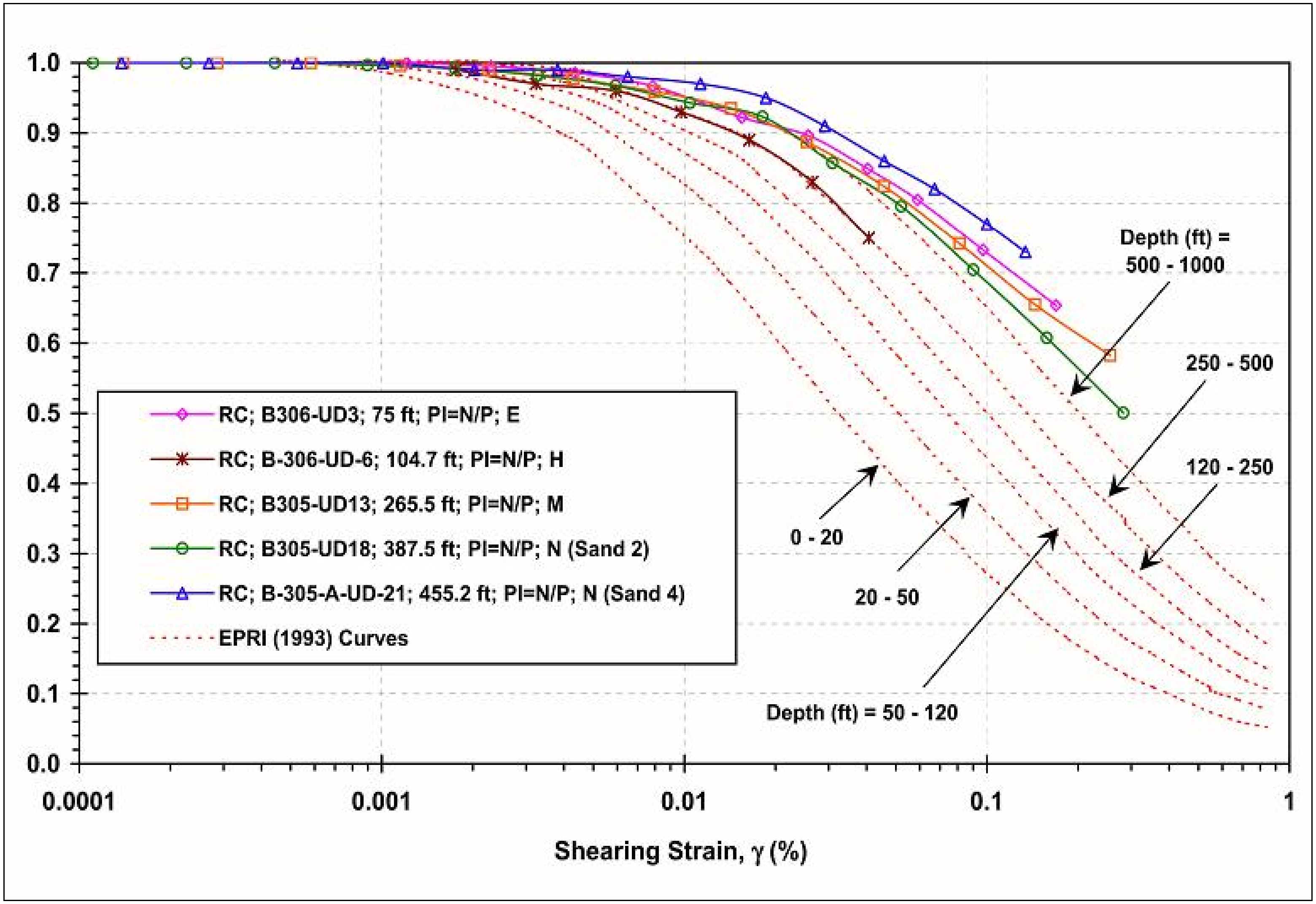

Figure 2.5.4-7. Site-Specific Sand Shear Modulus Curves Based on RCTS Testing (FSAR Figure 2.5S.4-62)

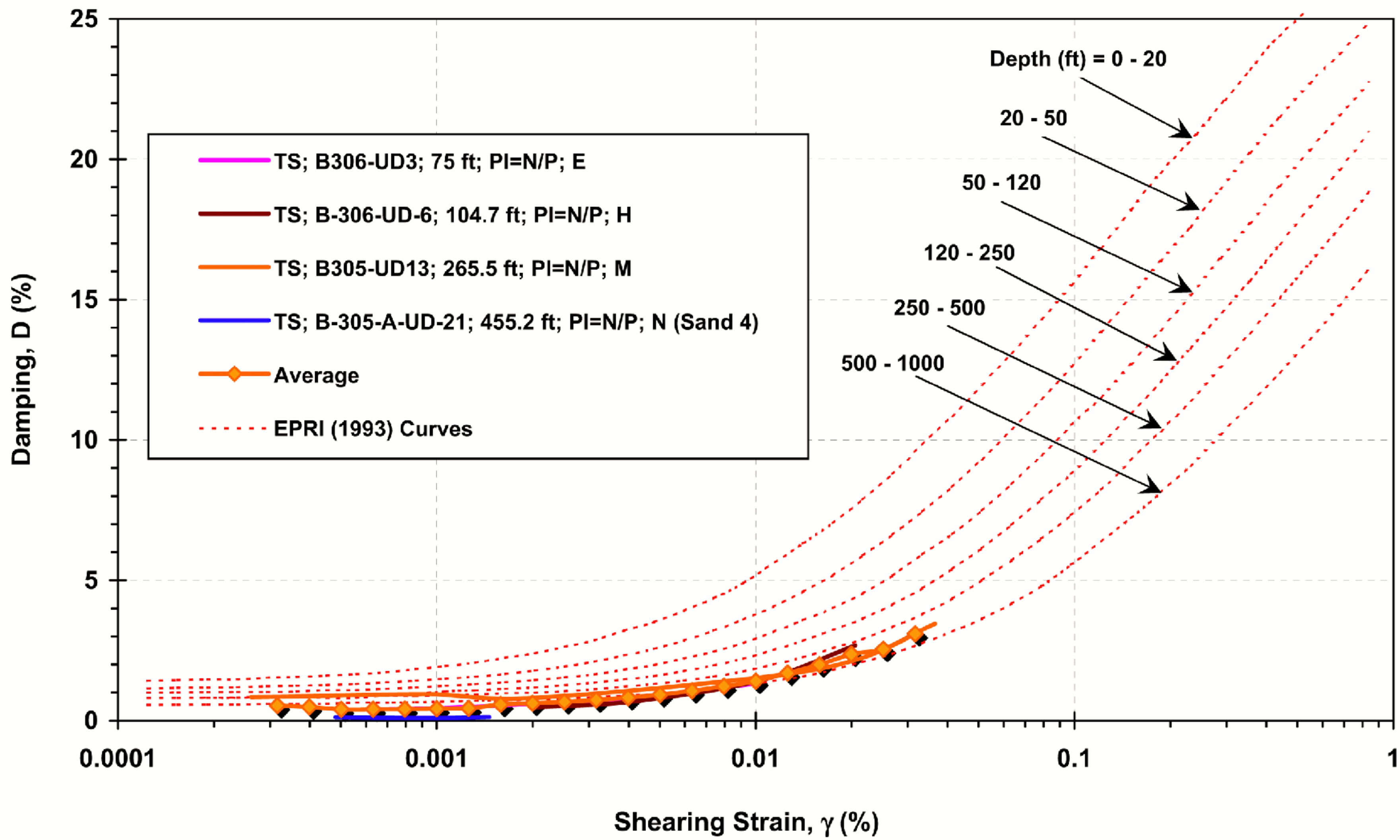

Figure 2.5.4-8. Site-Specific Sand Damping Ratio Curves Based on RCTS Testing (FSAR Figure 2.5S.4-66)

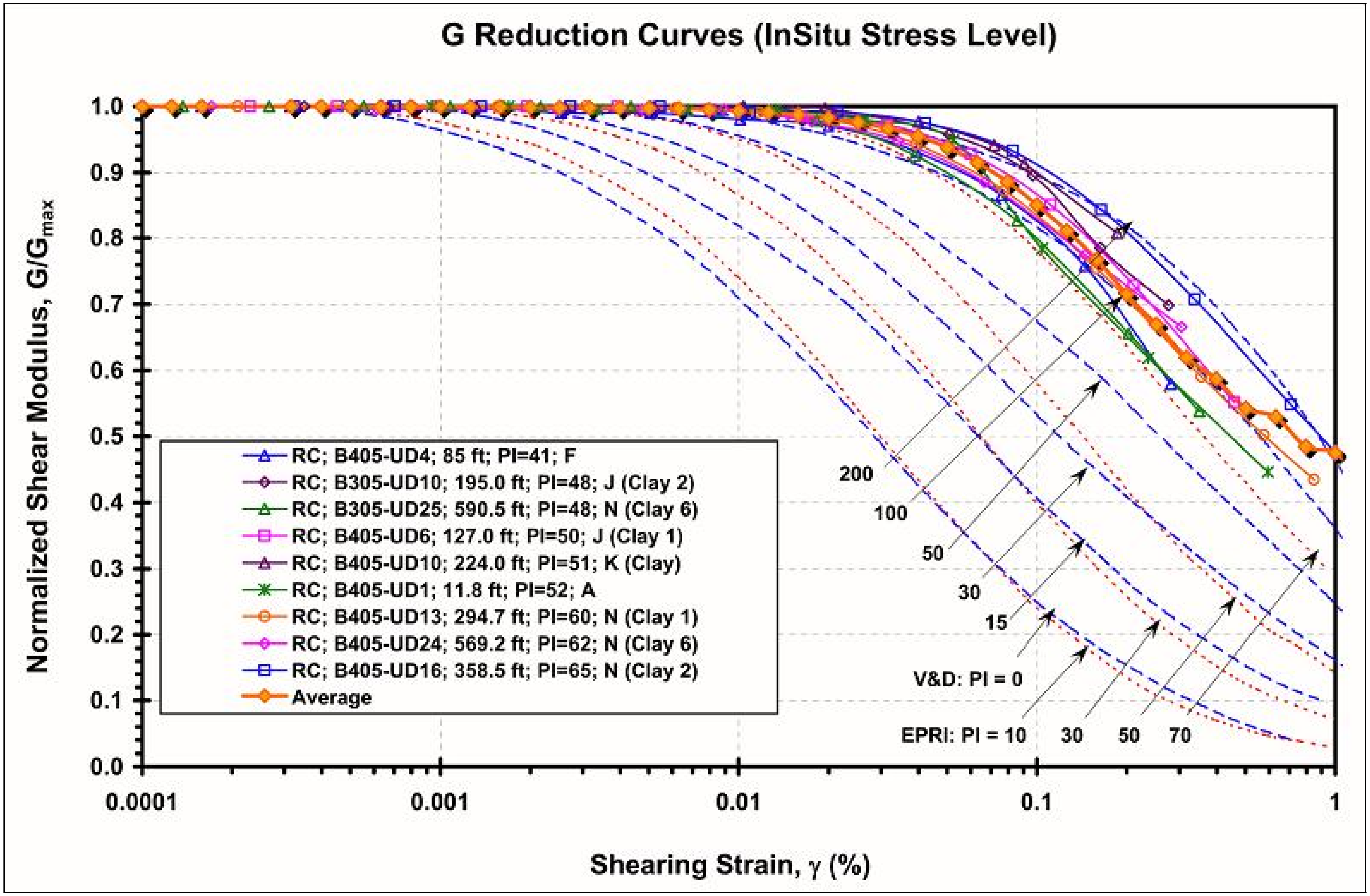

Figure 2.5.4-9. Shear Modulus Curves for High Plasticity Cohesive Soils Based on RCTS Testing (FSAR Figure 2.5S.4-64)

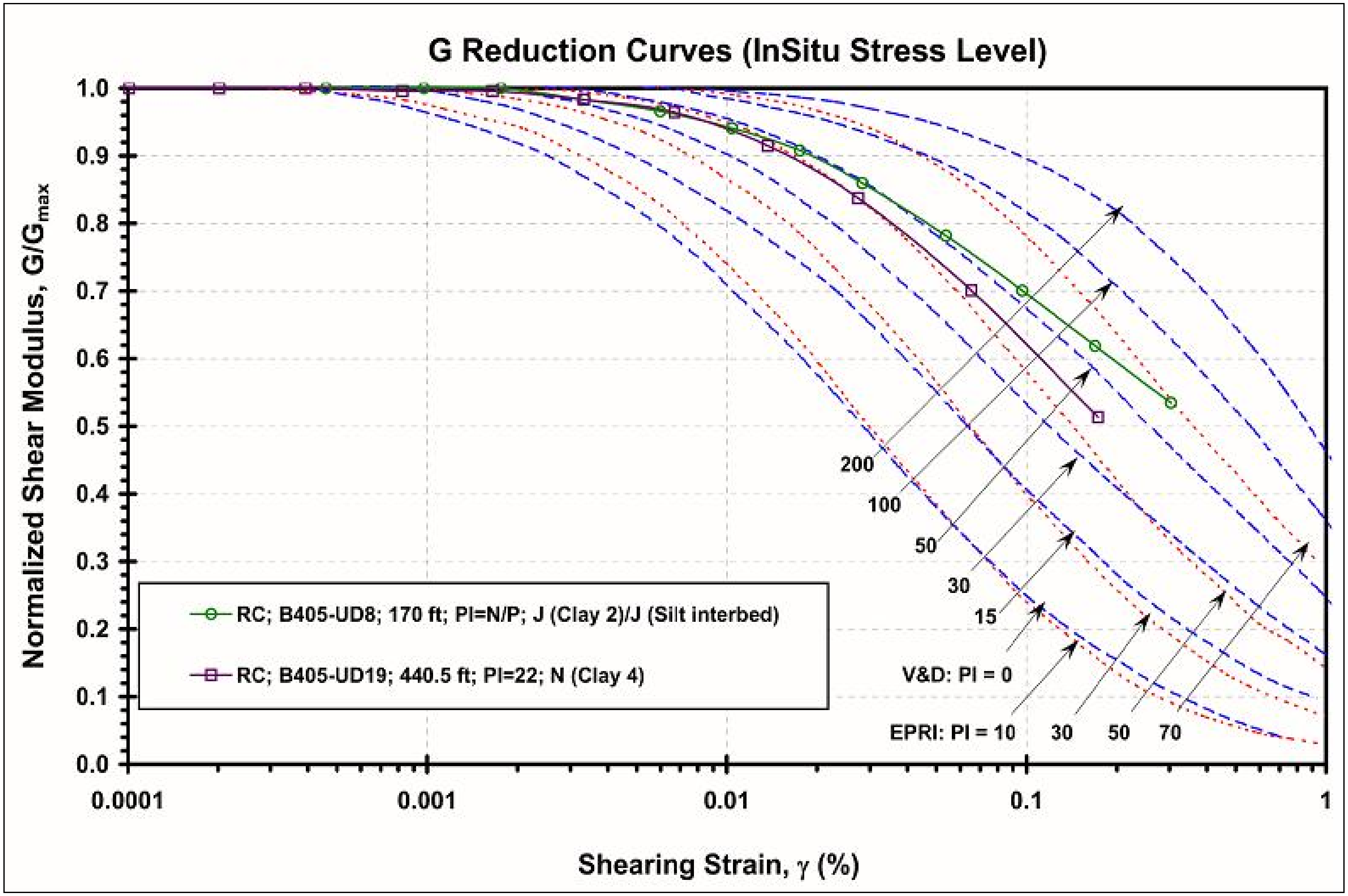

Figure 2.5.4-10. Shear Modulus Curves for Low Plasticity Cohesive Soils Based on RCTS Testing (FSAR Figure 2.5S.4-65)

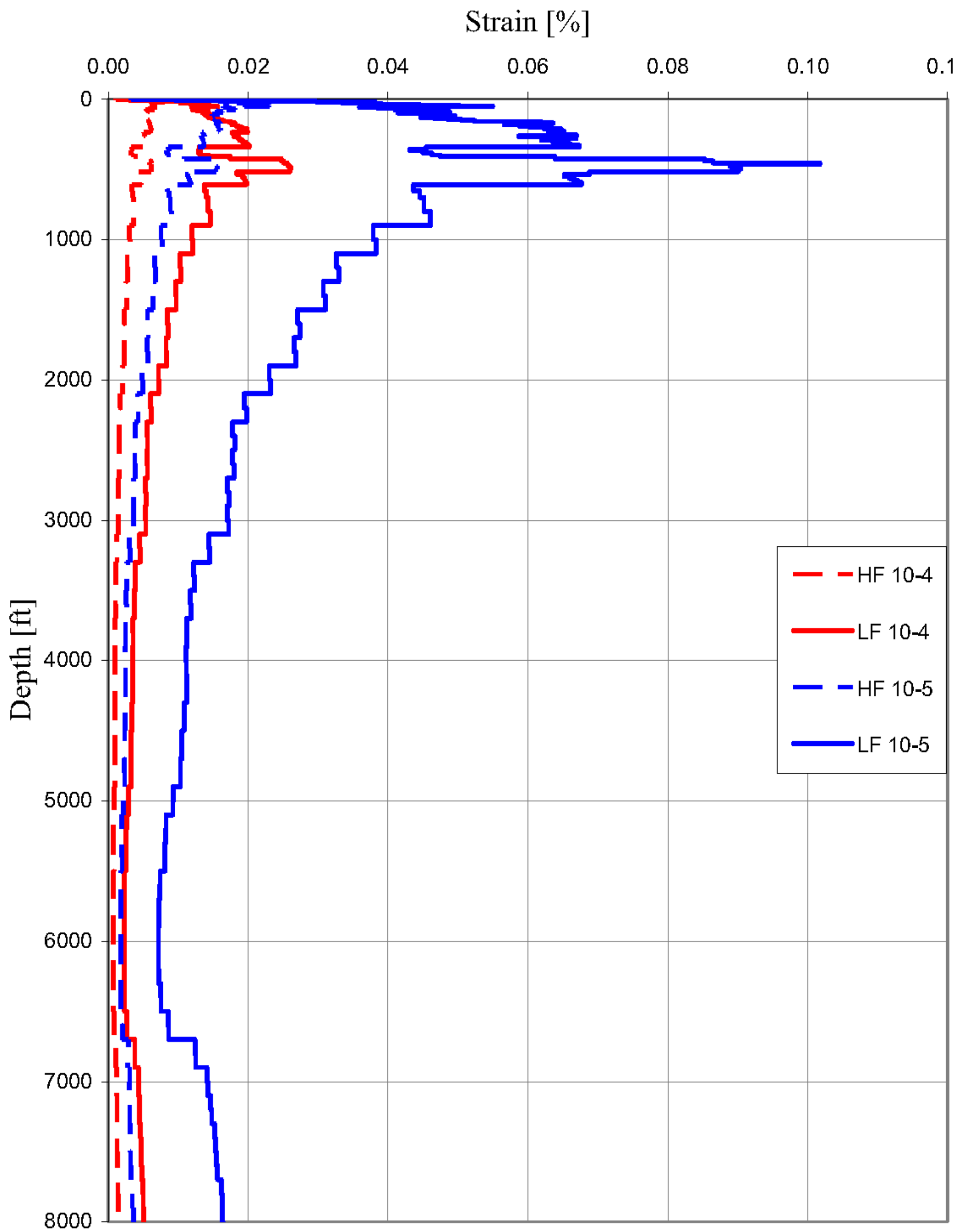

Figure 2.5.4-11. Logarithmic Mean Maximum Strain Profile (FSAR Figure 2.5S.2-47)

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

2.5.4.4.8 Liquefaction Potential

Although the ABWR DCD states that no liquefaction should occur within the STP site, FSAR Tables 2.5S.4-34 and 2.5S.4-35 show points of liquefaction potential within subsurface strata determined from SPT and CPT results. NRC 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.

The applicant's response to **RAI 02.05.04-5** dated July 9, 2008 (ML081960070), concludes 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 states 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.

The applicant's response to **RAI 02.05.04-28** dated August 10, 2009 (ML092250658), includes an examination of all results from the SPT, CPT, and shear wave velocity measurements. The applicant shows 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 shows 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 demonstrates 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 states 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 submits 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 provides 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 2.5.4-12 (**RAI 02.05.04-28** response, Figure 50) 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 response to the structure instability and/or structure interaction following an SSE, the applicant states 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 estimates 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 concludes 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 2.5.4-8 (**RAI 02.05.04-28** response Tables 4, 5, and 6) 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.

The applicant also presents 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 notes 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 concludes 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 concludes 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.

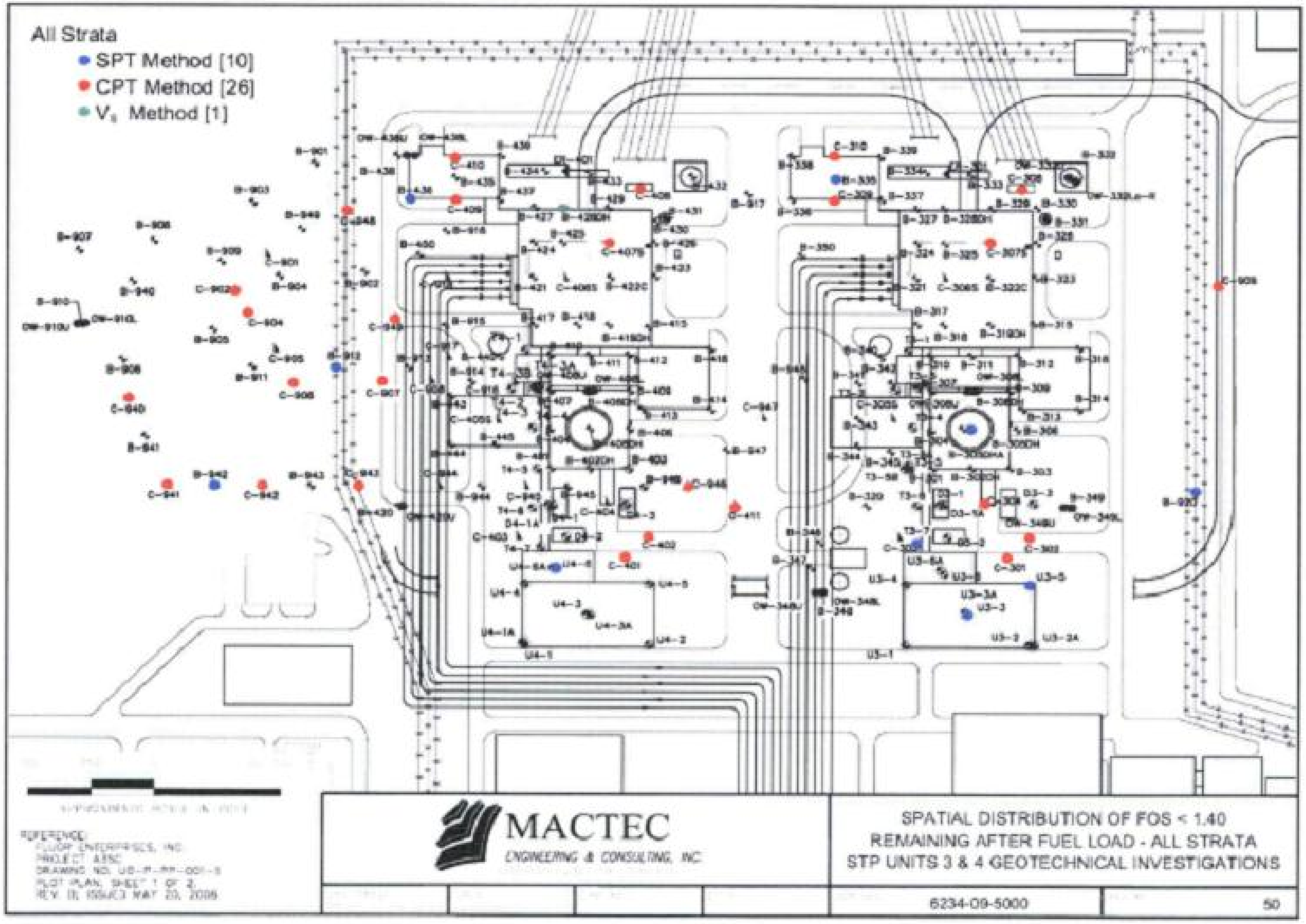

Figure 2.5.4-12. Plot of All Data Showing Plan Locations in All Strata of Soils Exhibiting a Factor of Safety Less Than 1.4 for Three Methods of Investigation (RAI 02.05.04-28 Response Figure 50)

Table 2.5.4-8. 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	FS	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	FS	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)

*CPT Method
Shear Wave method
FS – factor of safety

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 2.5.4-1 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 FS 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 conservativism 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. Accordingly, **RAI 02.05.04-28** and **RAI 02.05.04-5 are resolved**.

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.

2.5.4.4.9 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 2.5.2.4.6 includes a detailed evaluation of FSAR Subsection 2.5S.4.9.

2.5.4.4.10 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

NRC 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.

The applicant's response to **RAI 02.05.04-15** dated January 28, 2009 (ML090300648), provides 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 cites 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.

The applicant's response to **RAI 02.05.04-29** dated September 21, 2009 (ML092710096), states 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 response.

The applicant justifies 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 notes that RG 1.198 specifies 1.4 as an acceptable factor of safety for soil liquefaction, which the applicant concludes is acceptable because soil bearing and soil liquefaction are similar in importance with respect to foundation stability. Furthermore, the applicant states 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 states 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 concludes 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. Accordingly, **RAI 02.05.04-15** is resolved. 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.

The applicant's response to **RAI 02.05.04-35** dated April 27, 2010 (ML101270284), revises 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. Although the applicant did 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 concludes 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 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. Thus, **RAI 02.05.04-29** and **RAI 02.05.04-35** are resolved. Verification of the applicant's proposed changes to the FSAR is being tracked as **Confirmatory Item 02.05.04-35**.

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 presents the settlement predictions for total and differential settlement and tilt.

In FSAR Subsection 2.5S.4.10.4, the applicant describes the potential settlement for STP Units 3 and 4. However, it was not clear to NRC 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.

The applicant's response to **RAI 02.05.04-13** dated January 28, 2009 (ML091820695), clarifies 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 notes 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 2.5.4-9 (FSAR Section2.5S.4.10.4) shows the estimated differential settlement and the angular distortion/tilt values.

Table 2.5.4-9. 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
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

The applicant plans to mitigate the differential settlement by superstructure and mat rigidity. The applicant estimates 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 describes 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 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. Accordingly, **RAI 02.05.04-13** is resolved.

In a letter dated December 20, 2007 (ML073580003), the applicant states 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.

The applicant's response to **RAI 02.05.04-21** dated April 1, 2009 (ML090930717), adds 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.

The applicant's response to **RAI 02.05.04-30** dated September 21, 2009 (ML092710096), states 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 states 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.

The applicant's response to **RAI 02.05.04-34** dated April 27, 2010 (ML101270284), and **RAI 02.05.04-36** dated June 3, 2010 (101590397), revises 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 2.5.4-10 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 2.5.4-10 ITAAC for Settlement Prior to Fuel Load (COL Application Part 9, Table 3.0-13)

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. Accordingly, **RAI 02.05.04-21** and **RAI 02.05.04-30** are resolved.

Lateral Earth Pressures

NRC 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 and 2.5S.4-77 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 commits 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.

The applicant's response to **RAI 02.05.04-14** dated January 28, 2009 (ML090300648), provides 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. According, **RAI 02.05.04-14** is resolved.

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, (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 structures. RAI 0**2.05.04-29,** which referred to the dynamic bearing capacity of all the Seismic Category 1 structures, 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. NRC 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

NRC 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.

2.5.4.4.11 Design Criteria

NRC 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.

2.5.4.4.12 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.

2.5.4.5 *Post Combined License Activities*

The applicant identifies the following commitment:

- Commitment (COM 2.5S-3) – The applicant commits to 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 in SER Table 2.5.4-10. Furthermore, the applicant provides a three-part Backfill ITAAC as indicated in the SER Table 2.5.4-8, which addresses backfill properties, shear wave velocity and engineering properties of backfill.

2.5.4.6 *Conclusion*

NRC 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 stability of subsurface materials and foundations. With the exception of **Open Item 02.05.04-37** and **Confirmatory Items 02.05.04-33, 02.05.04-34, and 02.05.04-35**, no additional information is expected to be addressed in the COL FSAR related to this section.

The staff reviewed the STP 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 concluded 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 concluded 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 concluded 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 concludes that the computed factors of safety are adequate. The applicant has not identified the backfill source(s) and all of the information relevant to ensure 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 concluded 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 concluded that the stability of subsurface materials and foundations at the COL site are in accordance with the requirements of 10 CFR 100.23.

However, as a result of the above **open and confirmatory items**, the staff was unable to finalize the conclusions relating to the stability of subsurface materials and foundation, in accordance with the NRC requirements.

REFERENCES

American Concrete Institute, ACI 349, “Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary,” 2006.

American Society of Civil Engineers, Editing Board and Task Groups of the Committee on Nuclear Structures and Materials of the Structural Division, “Structural Analysis and Design of Nuclear Plant Structures,” ASCE 1980.

American Society of Civil Engineers, ASCE 4-98, “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” 2000.

American Society for Testing and Materials (ASTM) International, ASTM D 5778, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils,” 1995.

American Society for Testing and Materials (ASTM) International, ASTM D 1586, “Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils,” 1999.

American Society for Testing and Materials (ASTM) International, ASTM D 1587, “Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes,” 2000.

American Society for Testing and Materials (ASTM) International, ASTM D 4220, “Standard Practices for Preserving and Transporting Soil Samples,” 2000.

American Society for Testing and Materials (ASTM) International, ASTM D 4972, “Standard Test Method for pH of Soils,” 2001.

American Society for Testing and Materials (ASTM) International, ASTM D 422, “Standard Test Method for Particle Size Analysis of Soils,” 2002.

American Society for Testing and Materials (ASTM) International, ASTM D 1557, “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 feet-lbf/feet3 [2,700 kN-m/m3]),” 2002.

American Society for Testing and Materials (ASTM) International, ASTM D 4044, “Standard Test Method (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers,” 2002.

American Society for Testing and Materials (ASTM) International, ASTM D 2850, “Standard Test Method for Unconsolidated Undrained Triaxial Compression Test on Cohesive Soils,” 2003.

American Society for Testing and Materials (ASTM) International, ASTM D 4767, “Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils,” 2004.

American Society for Testing and Materials (ASTM) International, ASTM D 6913, “Standard Test Method for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis,” 2004.

American Society for Testing and Materials (ASTM) International, ASTM D 2435, “Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Incremental Loading,” 2004.

American Society for Testing and Materials (ASTM) International, ASTM D 3080, “Standard Test Method for Direct Shear Test of Soil Under Consolidated Drained Conditions,” 2004.

American Society for Testing and Materials (ASTM) International, ASTM D 5092, “Standard Practice for Design and Installation of Ground Water Monitoring Wells,” 2004.

American Society for Testing and Materials (ASTM) International, ASTM D 1883, “Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils,” 2005.

American Society for Testing and Materials (ASTM) International, ASTM D 4633, “Standard Test Method for Energy Measurement for Dynamic Penetrometers,” 2005.

American Society for Testing and Materials (ASTM) International, ASTM D 2216, “Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass,” 2005.

American Society for Testing and Materials (ASTM) International, ASTM D 4318, “Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils,” 2005.

American Society for Testing and Materials (ASTM) International, ASTM D 854, “Standard Test Method for Specific Gravity of Soil Solids by Water Pycnometer,” 2006.

American Society for Testing and Materials (ASTM) International, ASTM D 2113, “Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation,” 2008.

American Society for Testing and Materials (ASTM) International, ASTM D 2166, “Standard Test Method for Unconfined Compressive Strength of Cohesive Soils,” 2006.

A. W. Bishop, “The use of slip circles in the stability analysis of earth slopes,” *Geotechnique*, 5:1, 7–17, 1955.

J.E. Bowles, 1997, “Foundation Analysis and Design, (4th edition),” 1997.

J.R. Davie and M.R. Lewis, “Settlement of Two Tall Chimney Foundations,” Proceedings of the 2nd International Conference on Case Histories in Geotechnical Engineering, St. Louis, MO, 1309–1313, 1988.

Electric Power Research Institute (EPRI), EPRI Report No. TR-102293, “Guidelines for Determining Design Basis Ground Motions,” Volumes 1-5, Appendix 7.A, “Modeling of Dynamic Soil Properties,” 1993.

Electric Power Research Institute (EPRI) Technical Report No. 1009684, “CEUS Ground Motion Project Final Report,” EPRI, Dominion Energy, Entergy Nuclear, and Exelon Generation Company, December 2004.

H. Garry and P.E. Gregory, “Slope Stability Analysis System,” GSTABL7, GSTABL7 with STEDwin software, version 2.004, June 2003.

Gulla et al., “Effect of Weathering on the Compressibility and Shear Strength of a Natural Clay,” *Canadian Geotechnical Journal,* 43:6, 618–625, 2006.

K. Ishihara and M. Yoshimine, “Evaluation of Settlements in Sand Deposits following Liquefaction during Earthquakes,” *Soils and Foundations*, 32:1, 173–188, 1992.

J. Jaky, “Pressure in Soils,” *Proceedings of 2nd Int. Conf. on Soil Mech. and Found. Eng*., Rotterdam, 1:103–107, 1948.

N. Janbu, “Application of Composite Slope Surface for Stability Analysis,” Proceedings of the European Conference on Stability of Earth Slopes, 3:43–49, Stockholm, 1954.

L.J. Mahar and M. O’Neil, “Geotechnical Characterization of Desiccated Clay,” *Journal of Geotechnical Engineering*, 109:1, 56–71, January 1983.

G. Mesri, and M. Shahien, “Residual Shear Strength Mobilized in First-Time Slope Failures,” *Journal of Geotechnical Engineering*, 129:(1), 2003.

Naval Facilities Engineering Manual DM 7.02, “Foundations and Earth Structures: NAVFAC DM 7.02,” 2009.

F. Ostadan, “Seismic Soil Pressure for Building Walls—An Updated Approach,” The 11th International Conference on Soil Dynamics and Earthquake Engineering (11th ICSDEE) and the 3rd International Conference on Earthquake Geotechnical Engineering (3rd ICEGE), 2004.

A.W. Skempton, “Long Term Stability of Clay Slopes,” *Geotechnique*, 14:2, 77–101, 1964.

A.W. Skempton, “First-Time Slides in Over-Consolidated Clays,” Technical Notes, *Geotechnique*, 20, 320–324, 1970.

A.W. Skempton, “Slope Stability of Cuttings in Brown London Clay,” *Proceedings of the International Conference on Soil Mechanics and Foundation Engineering*, 9, 261–269, 1977.

K.H. Stokoe, et al., “Test Procedures and Calibration Documentation Associated with the RCTS and URC Tests at the University of Texas at Austin,” Geotechnical Engineering Report GR04-6, Geotechnical Engineering Center, Civil Engineering Department, 2006.

U.S. Environmental Protection Agency, Report No. EPA/600/R-93/100, EPA 300.0, “Method for the Determination of Inorganic Substances in Environmental Samples,” 1993.

M. Vucetic and R. Dobry, “Effect of Soil Plasticity on Cyclic Response,” *Journal of Geotechnical Engineering*, 117:1, January 1991.

T.L Youd, et al., “Liquefaction Resistance of Soils: Summary Report from the 1996 National Center for Earthquake Engineering Research (NCEER) and the 1998 NCEER/ National Science Foundation (NSF) Workshops on Evaluation of Liquefaction of Soils,” *Journal of Geotechnical and Environmental Engineering*, 127:10, October 2001

2.5.5 Stability of Slopes

2.5.5.1 *Introduction*

FSAR Section 2.5S.5 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

2.5.5.2 *Summary of Application*

The applicant provides supplemental information to address COL License Information Items 2.40 and 2.41, as specified in Section 2.3 of the ABWR DCD.

COL License Information Items

- 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 states that the required information is in Section 2.5S.5.

- 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 states that there "are no embankments or dams that impound water required for safe operation (and shutdown)."

The applicant developed FSAR Section 2.5S.5 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 Sections 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.

2.5.5.2.1 Slope Characteristics

In FSAR Subsection 2.5S.5.1, the applicant describes the characteristics of existing permanent slopes. The applicant states 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 2.5.5-1 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 states that the interior embankment slopes are 2.5H:1V, while the exterior slopes are 3H:1V.

2.5.5.2.2 Design Criteria and Analysis

In FSAR Subsection 2.5S.5.2, the applicant summarizes 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 notes 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 2.5.5-1 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 states 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.

2.5.5.2.3 Boring Logs

In FSAR Subsection 2.5S.5.3, the applicant refers 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 provides the logs of borings and information related to field testing corresponding to STP Units 3 and 4 subsurface investigations in a MACTEC report (2007).

2.5.5.2.4 Compacted Fill

The applicant states that FSAR Subsection 2.5S.4.5 addresses the compacted fill requirements.

2.5.5.3 *Regulatory Basis*

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 structures, systems, and components (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.

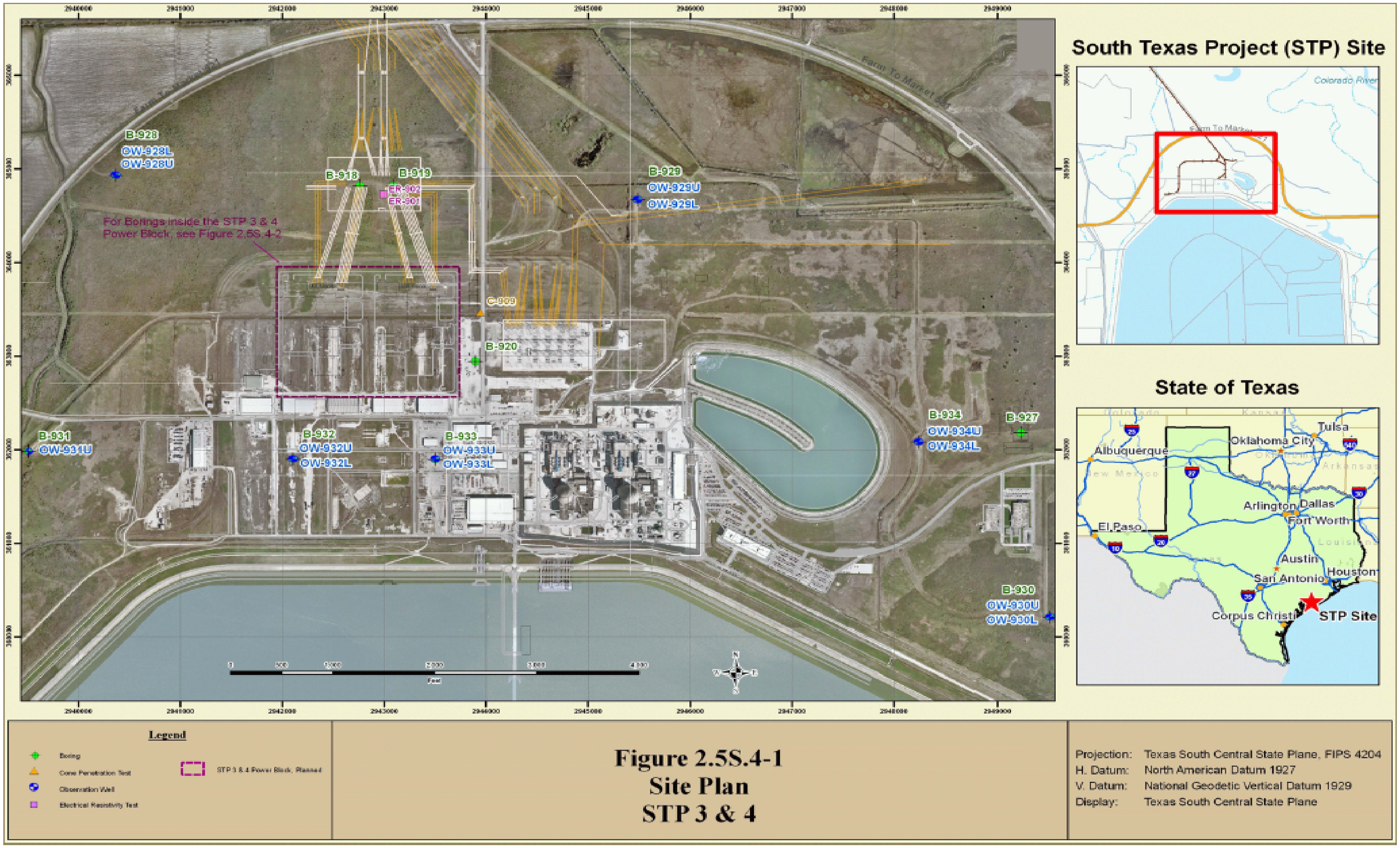

Figure 2.5.5-1. Site Plan including Location of the Main Cooling Reservoir Embankment (FSAR Figure 2.5S.4-1)

Table 2.5.5-1. Slope Stability Analysis Considerations

CASE	FACTOR OF SAFETY
1. Reservoir water level set at maximum operating level	1.7 to 1.8 (exterior slopes) 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) 1.7 to 4.7 (SSE)

- 10 CFR Part 50, Appendix A, General Design Criterion 1 (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. 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, "Design bases for protection against natural phenomena," 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, "Cooling water," 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, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants,“ 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, “Earthquake Engineering Criteria for Nuclear Power Plants,” applies to the design of nuclear power plant SSCs 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 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:

(1) Slope Characteristics: 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.

(2) 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.

(3) 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.

(4) 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:

- Regulatory Guide 1.27, "Ultimate Heat Sink for Nuclear Power Plants"

- Regulatory Guide 1.28, "Quality Assurance Program Requirements (Design and Construction)"

- Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants"

- Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants"

- Regulatory Guide 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites"

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

2.5.5.4 *Technical Evaluation*

NRC 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

NRC 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 states 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 references 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 2.5.5-1. 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 Unit 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 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.

2.5.5.4.1 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, NRC 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 refers 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.

2.5.5.4.2 Design Criteria and Analysis

NRC 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.

2.5.5.4.3 Boring Logs

NRC 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.

2.5.5.4.4 Compacted Fill

NRC staff's evaluation of compacted fill is in Subsection 2.5S.4.4 of this SER.

2.5.5.5 *Post Combined License Activities*

There are no post COL activities related to this section.

2.5.5.6 *Conclusion*

As discussed above, the applicant presents and substantiates information that establishes the stability of all earth and rock slopes, both natural and manmade, at the plant site. NRC 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 concluded that the STP Units 3 and 4 site is suitable with respect to the criteria governing the stability of slopes.