2.5 Geology, Seismology, and Geotechnical Engineering

In Section 2.5, "Geology, Seismology, and Geotechnical Engineering," of the SSAR, SERI provided a detailed description of the geological, seismological, and geotechnical engineering properties of the ESP site. In this SER, Section 2.5.1, "Basic Data," describes basic geological and seismological data, especially the data published since 1986 for the area within a 200-mile radius of the ESP site, and presents updated seismic sources. Section 2.5.2, "Vibratory Ground Motion," evaluates the vibratory ground motion for the ESP site and analyzes the safe-shutdown earthquake (SSE) ground motion. Section 2.5.3, "Surface Faulting," describes the potential for surface faulting at or near the surface of the ESP site. Section 2.5.4, "Stability of Subsurface Materials and Foundations," presents the results of site geotechnical investigations and discusses the stability of subsurface materials and foundations. Section 2.5.5, "Stability of Slopes," defers the analyses for slope stability at the site and dam performance, respectively, to the COL stage. Finally, Section 2.5.6, "Embankments and Dams," briefly states that no embankments exist within the site location (1 kilometer), and no impoundment structures exist within the site area (8 kilometers), which could affect the safety of the proposed new facility.

The applicant also stated in SSAR Section 2.5 of the ESP application that the UFSAR for GGNS formed the basis for its characterization of the site geology, seismology, and geotechnical engineering. As such, the material in Section 2.5 of the SSAR focuses on any newly published information since the publication of the GGNS UFSAR in the 1970s. In addition, the technical information presented in Section 2.5 of the ESP application is based largely on the applicant's surface and subsurface geological, seismological, geophysical, and geotechnical investigations that were performed in progressively greater detail as the location of the investigations neared the site. The applicant defined the following zones of investigation in terms of their distances from the ESP site, following the recommendation of RG 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," issued March 1997:

- site region—within 320 kilometers (200 miles)
- site vicinity—within 40 kilometers (25 miles)
- site area—within 8 kilometers (5 miles)
- site location—within 1 kilometer (0.6 miles)

The applicant adopted the Electric Power Research Institute (EPRI) report, "Seismic Hazard Methodology for the Central and Eastern United States," issued July 1986, to model regional seismic sources. Therefore, SSAR Section 2.5 focuses on those data developed since the publication of the 1986 EPRI report. RG 1.165 allows the applicant to use the seismic source interpretations developed by Savy, et al., at Lawrence Livermore National Laboratory (LLNL) in the "Eastern Seismic Hazard Characterization Update," issued 1993, or the EPRI document as inputs for a site-specific analysis.

2.5.1 Regional and Site Geology

Section 2.5.1 of the SSAR describes the regional and site geology for the ESP site. Sections 2.5.1.1 and 2.5.1.2 describe the general geologic, seismologic, and tectonic setting of the site region and site area, respectively.

2.5.1.1 Technical Information in the Application

2.5.1.1.1 Regional Geology

Section 2.5.1.1 of the ESP application describes the (1) regional physiography, (2) regional geological provinces, (3) regional geologic history, (4) regional stratigraphy, (5) regional tectonic settings, and (6) regional seismicity of the site region.

Regional Physiography. SSAR Section 2.5.1.1.1 describes the regional physiography. The ESP site is located within the Gulf Coastal Plain physiographic province. The following subprovinces make up this physiographic province, as shown in Figure 2.5.1-1:

- Loess Hills subprovince
- Mississippi Alluvial Valley subprovince
- Eastern Hills subprovince
- Western Hills subprovince
- Southern Hills subprovince
- Prairie Coastwise Terrace subprovince
- Chenier Plain subprovince
- Delta Plain subprovince

The proposed site is located within the Mississippi Alluvial Valley subprovince. This subprovince includes several interdistributary lowlands, basins, and ridges; elevations within the subprovince range from 50–250 feet. The topographic highs along the Mississippi River are remnants of older alluvial deposits that were mostly eroded and removed from the valley. The Mississippi Alluvial Valley is relatively flat with a gentle southward gradient and is characterized by fluvial geomorphic features typical of a braided stream and meandering river system.

Regional Geological Provinces. SSAR Section 2.5.1.1.2 describes the regional geological provinces. The Gulf Coast Plain physiographic province is divided into two primary geologic provinces, the Mississippi embayment and the Gulf Coast Basin. The site region is located within the Gulf Coast Basin, which includes the southern portion of the Mississippi embayment (see Figure 2.5.1-2).

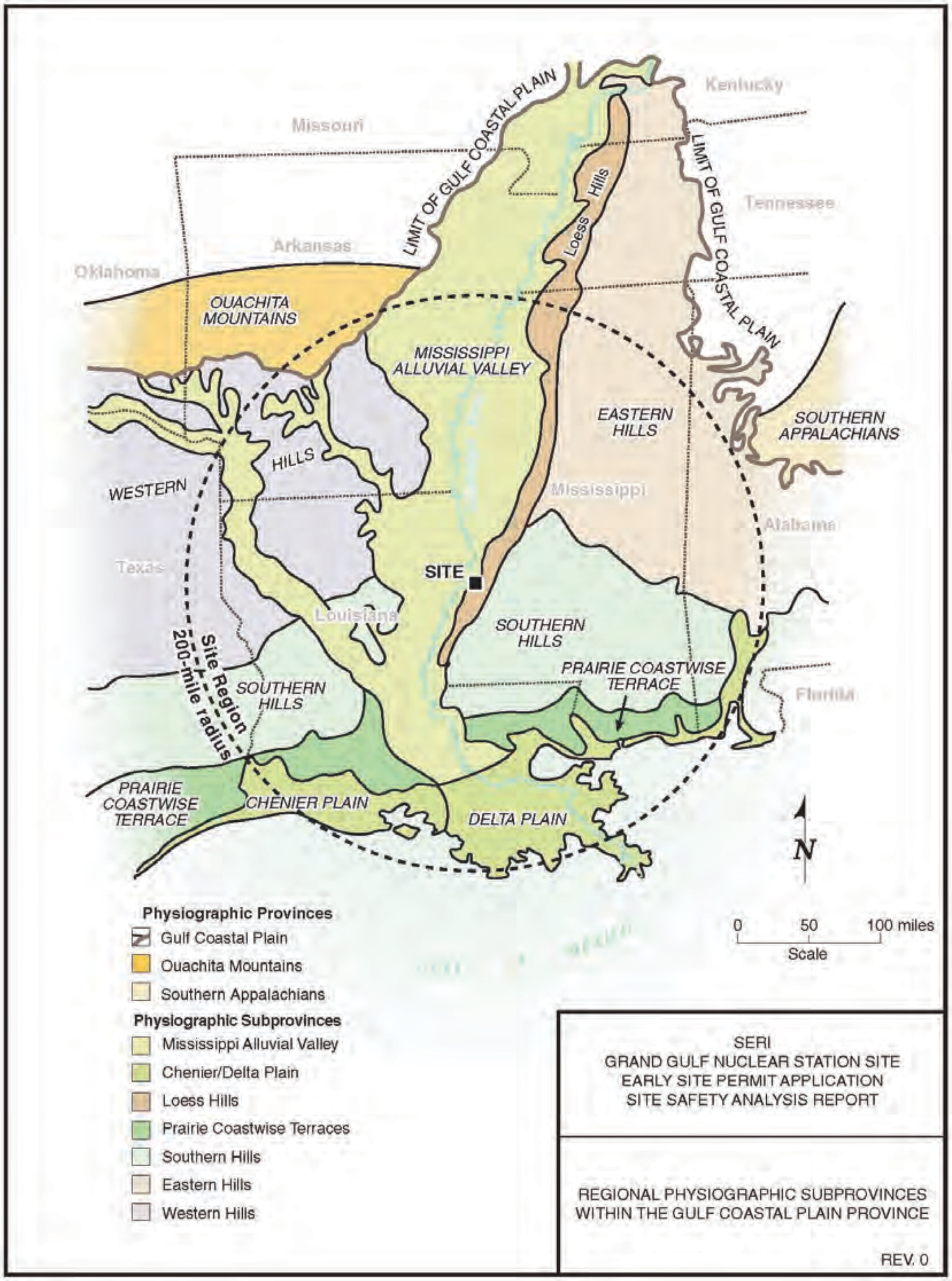


Figure 2.5.1-1 Physiographic subprovinces within the Gulf Coastal Plain province

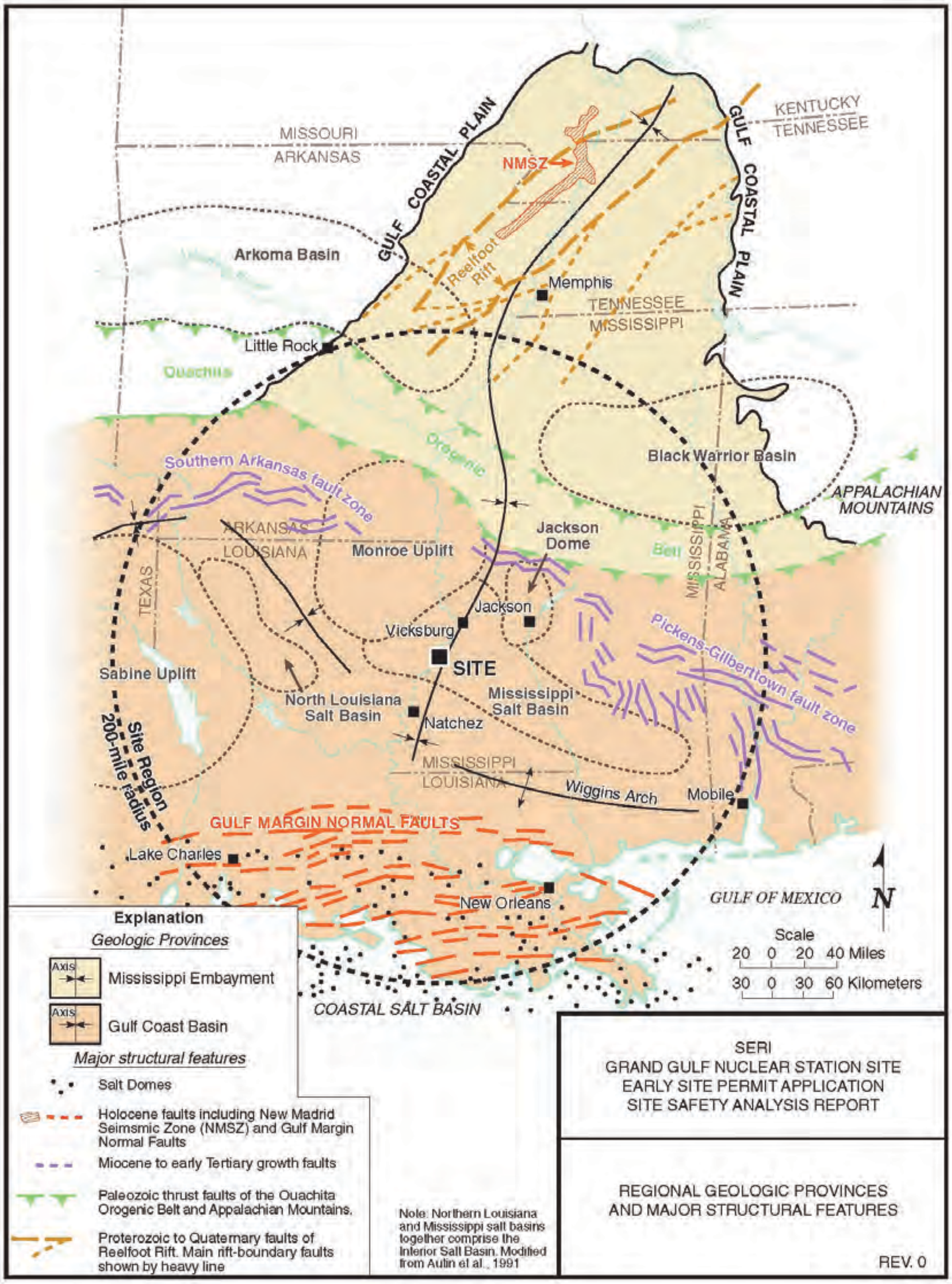


Figure 2.5.1-2 Geological provinces and major structural features

The Gulf Coast Basin extends from the Gulf of Mexico to the buried Ouachita Orogenic Belt. The basin formed during initial rifting of the Gulf of Mexico in the Triassic Period (248–209 million years before present (Ma)). The basement rocks within the Gulf Coast Basin are transitional between continental and oceanic materials, depending on the distance from the Gulf of Mexico. The Mississippi embayment lies between the Ouachita Mountains in southern Arkansas and the Appalachian Mountains in west-central Alabama, which cover the northern margin of the Gulf Coastal Plain. The Mississippi embayment formed during the late Cretaceous Period (144–66 Ma) as a result of crustal downwarping associated with the extension of the Reelfoot Rift. The embayment is structurally a south-southwest dipping syncline, underlain by Paleozoic strata and igneous and metamorphic basement rocks. This syncline extends to the Gulf Coast Basin and forms a structural downwarp that affects the depth to the basement and the thickness of the overlaying sedimentary column. The limbs of the syncline in the Gulf Coast Basin typically dip less than 1 degree towards the syncline axis. The largest historical earthquakes in the central and eastern United States (CEUS), the 1811–1812 New Madrid earthquakes, occurred inside the Mississippi embayment.

<u>Regional Geologic History</u>. SSAR Section 2.5.1.1.3 describes the regional geologic history of the ESP site. The applicant stated that the site region is located in the south-central margin of the North American craton. The crystalline basement of the North American craton in the central United States is wholly Precambrian in age (more than 570 Ma), with the possible exception of transitional basement rocks underlying the Gulf Coast Basin. There are eight major cratonic units, formed by orogenies ranging from Archean (3800 Ma) to middle Proterozoic (750 Ma). Overall, the North American craton enlarged gradually to the south and east because of lateral accretion during successively younger Precambrian orogenies. The similarity in the ages of the rift systems within the North American craton indicated that the Reelfoot Rift initiated as a failed arm of a triple junction (an intersection of three oceanic ridges) during an episode of late Precambrian continental fragmentation.

During the early Mesozoic Era, known as the Triassic Period (248–209 Ma), the Reelfoot Rift was reactivated, resulting in additional extension and intrusion. The Gulf of Mexico began forming during the Triassic Period from extensional rifting of the supercontinent Pangea and divergent motion of the North American and Afro-South American plates. The slow deposition of sediments on top of the Paleozoic (570–245 Ma) sedimentary rocks formed the Gulf Coastal Plain, to the north of the Gulf of Mexico. By the mid-Jurassic Period (180 Ma), the Gulf Coast region became a restricted seaway with evaporitic conditions that accumulated more than 9900 feet of salt deposits. The Mississippi embayment experienced deposition-erosion episodes during the Cretaceous Period (66–144 Ma). At the end of the Cretaceous Period, volcanic activity and igneous intrusions formed the volcanogenic structural highs, isolating the northern part of the embayment from the Gulf Costal Basin to the south.

During the Cenozoic Era (66 Ma–present), the Gulf Coastal Plain expanded southward by as much as 250 miles as the Mississippi River transported massive volumes of sediments to the Gulf Coast Basin during the Pleistocene Epoch (1.8–0.11 Ma). The rising sea level submerged the late Pleistocene continental shelf and reached its present position approximately 3000–4000 years ago, defining the current configuration of the Gulf Coast margin.

<u>Regional Stratigraphy</u>. SSAR Section 2.5.1.1.4 describes the stratigraphy from the youngest to the oldest for the ESP site region. The applicant described the rocks and sediments in the site

region in great detail, including their distribution, components, and environment. The geological map shown in Figure 2.5.1-3 outlines the stratigraphic exposures in the site region.

Cenozoic Era (66 Ma–present)

The Cenozoic Era consists of Quaternary (1.8 Ma–present) and Tertiary Periods (66–1.8 Ma). Deposits from these two periods, especially the Quaternary, form the current surface configuration around the ESP site.

Quaternary deposits within the site region occur along the Mississippi Alluvial Valley and its tributaries, the Southern Hills subprovince of the Gulf Coastal Plain, and the Loess Hills subprovince. Holocene (0.11 Ma–present) deposits include alluvium and loess that occur within the Mississippi River Valley and its tributary valleys, and deltaic and beach facies within the Chenier Plain and Delta Plain. Holocene alluvial and deltaic deposits thicken from a few tens of feet in the northern portion of the site region to greater than 600 feet in the southern portion of the site region. In the site vicinity, the Holocene deposits in the Mississippi Alluvial Valley range from 0–400 feet thick. Holocene sediments in the two main tributary valleys within the site vicinity, Bayou Pierre and Big Black River, range in thickness from 70–100 feet. Pleistocene deposits in the site region consist of mainly loess and terrace deposits. Pleistocene loess occurs along the eastern edge of the Mississippi Alluvial Valley in a belt 10–30 miles wide. The maximum thickness of the loess is 75 feet. Erosion along the eastern side of the Mississippi floodplain forms a prominent erosional escarpment in the loess. Loess deposits unconformably overlie the Pleistocene to Pliocene alluvial deposits and Tertiary deposits in the site vicinity and site area. Pleistocene terrace deposits occur along most of the Mississippi Alluvial Valley and extend across the site region. The thickness of the terrace deposits also varies significantly, depending on the location. The applicant stated that the stratigraphic continuity and absence of vertical deformation of the terraces demonstrate the tectonic stability of the Gulf Coastal Plain through the Pliocene and Pleistocene Epochs.

Tertiary deposits are more than 6000 feet thick in the site vicinity. These deposits thicken from north to south across the region with a maximum thickness of more than 50,000 feet in the Gulf of Mexico. The Tertiary deposits consist of terrigenous sediment eroded from the interior of North America and marine sediment deposited during marine transgressions and regressions. Among the different series of sedimentary deposits in the Tertiary Period, the Miocene Catahoula formation is one of the most extensive deposits in the site vicinity. The applicant stated that this formation underlies the site area and is the load-bearing layer for the existing GGNS and the potential new facility.

Mesozoic Era (245–66 Ma)

Most of the Mesozoic deposits in the site region are located underneath the surface, with the exception of some locally exposed Cretaceous marine and terrestrial sediments that accumulated in response to active rifting and marine transgressions and regressions. Deposits of the Cretaceous system are distributed in the eastern and northern portions of the site region. The Cretaceous system has a maximum combined thickness of more than 5000 feet beneath the site and mainly includes chalk, clay, sand, limestone, and marl. Jurassic system deposits in the site region include anhydrite, sandstone, conglomerate, limestone, shale, and sandstone.

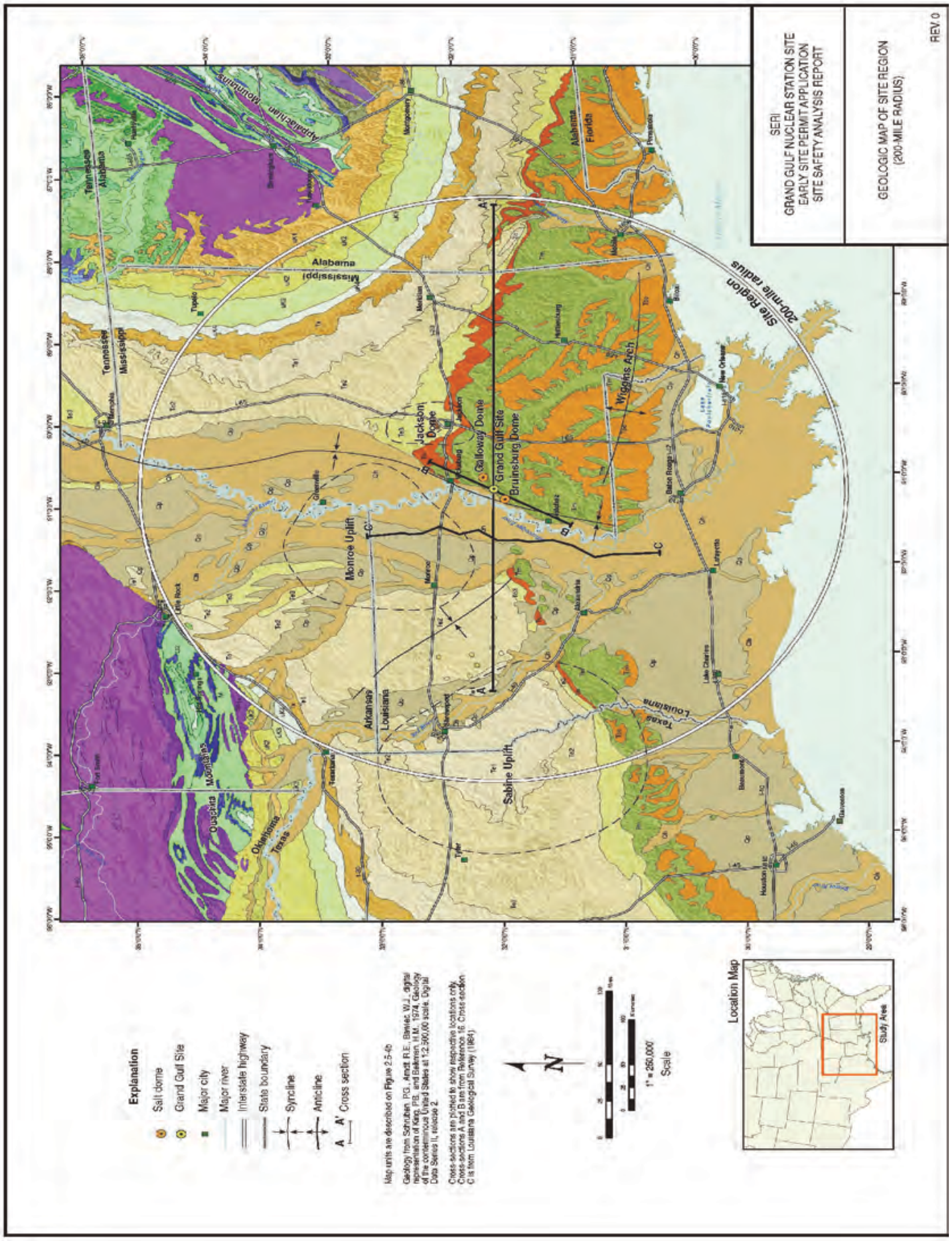


Figure 2.5.1-3 Geological map of the site region

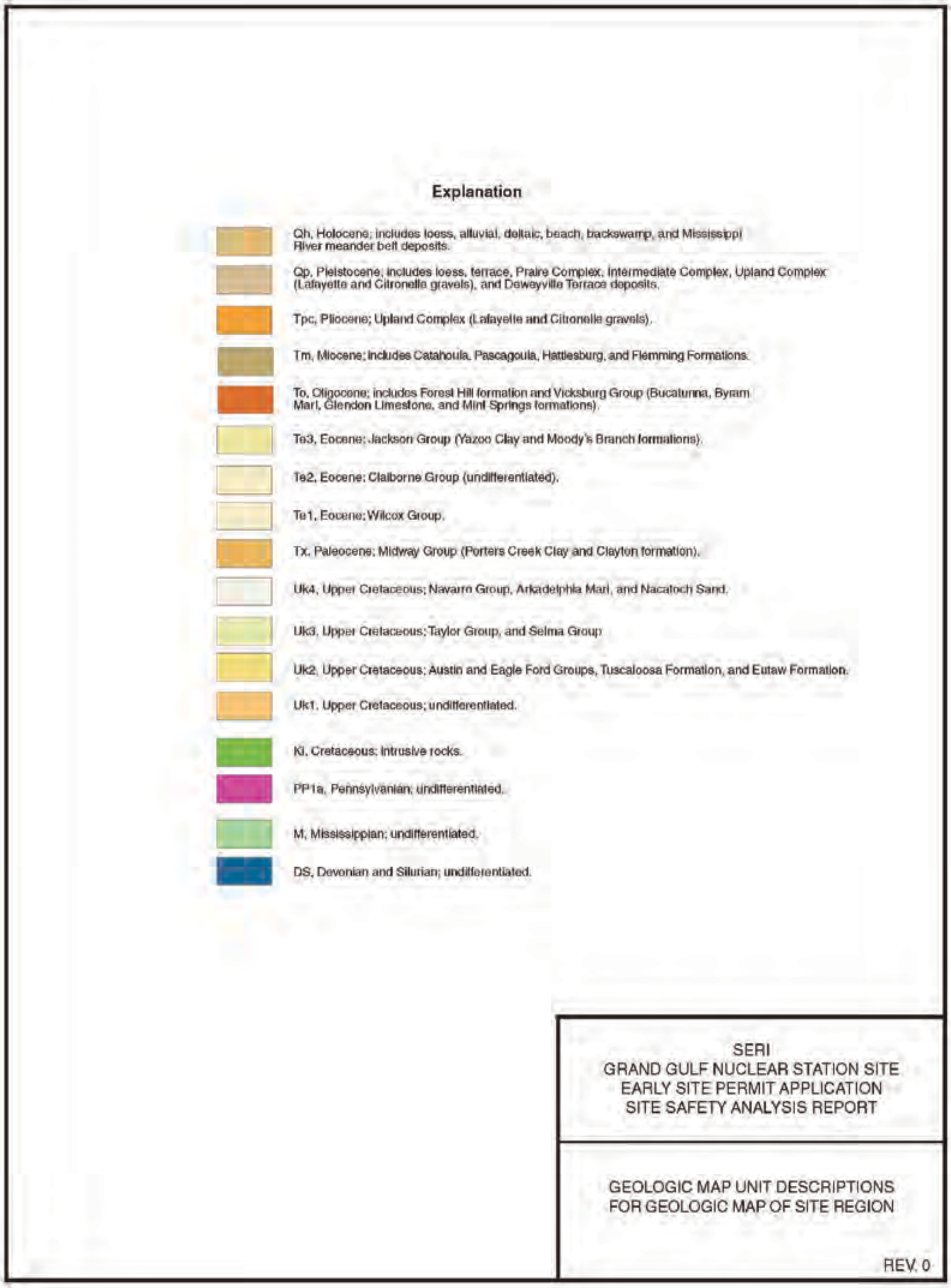


Figure 2.5.1-3 (cont.) Geological map of the site region

Thick evaporite deposits such as salt also exist in the Interior Salt Basin and Coastal Salt Basin and caused widespread diapirism and associated folding and faulting. Cumulatively, the Jurassic deposits in the southern portion of the site region have a thickness of nearly 10,000 feet. The only identified Triassic deposits in the subsurface of the site region consist of shales, mudstones, and siltstones, as well as fine-grained sandstone.

Paleozoic Era (245–570 Ma)

Paleozoic rocks are exposed in the northwestern portion of the site region. Deposits of the Paleozoic Era beneath the site consist of 7 major stratigraphic series and 19 individual formations. The maximum combined thickness of Paleozoic deposits is in excess of 5600 feet in the site region and unknown in other regions. The depth to these deposits beneath the site vicinity is greater than 13,000 feet. Deposits of the Mississippian (362–322 Ma) and Pennsylvanian (322–290 Ma) Periods consist of interbedded shale, fine-grained sandstone, and minor limestone. Deposits of the Ordovician (510–439 Ma) Period consist of dolomite interbedded with thin layers of limestone, shale, and sandstone.

Precambrian (more than 570 Ma)

Based on deep oil and gas exploration wells in the site vicinity, the depth to Precambrian basement rocks is 6–8 miles. The applicant stated that thick, younger sedimentary layers in the Gulf Coast Basin have obstructed the collection of rock samples from the Precambrian basement.

Regional Tectonic Setting. SSAR Section 2.5.1.1.5 describes the regional tectonic setting of the ESP site. The applicant summarized the general tectonic framework and presented the orientation of tectonic stresses in the site region. The applicant also described individual seismic source zones, not only those identified by the 1986 EPRI seismic hazard model, but also new seismic source zones recognized after the 1986 EPRI study. The south-central United States, where the ESP site is located, is a passive continental margin with no nearby plate motion between the Gulf of Mexico and the oceanic plate and the North America continental plate. As such, the region has low earthquake activity and low stress and is typical of a stable continental region. The horizontal compressive tectonic stress in the CEUS orients primarily northeast-southwest, caused by ridge push associated with the mid-Atlantic oceanic ridge. In contrast to the midcontinent compression stress, the southward-oriented extension along the Gulf Coast reflects crustal loading and deformation within the Mississippi River deltaic complex in the Gulf of Mexico. This extension may be distinct from the regional east-northeastward-directed compressive stress in the underlying basement rock. The primary tectonic elements of the region are ancient rift systems, such as the Reelfoot Rift, or former collision zones, such as the Ouachita Orogenic Belt (see Figure 2.5.1-4). Younger tectonic activity appears to be entirely related to reactivation of the rift structures or collision zones. As such, a majority of the seismic events with body-wave magnitude (m_b) greater than 4.5 are concentrated inside the Reelfoot Rift and the Ouachita Orogenic Belt.

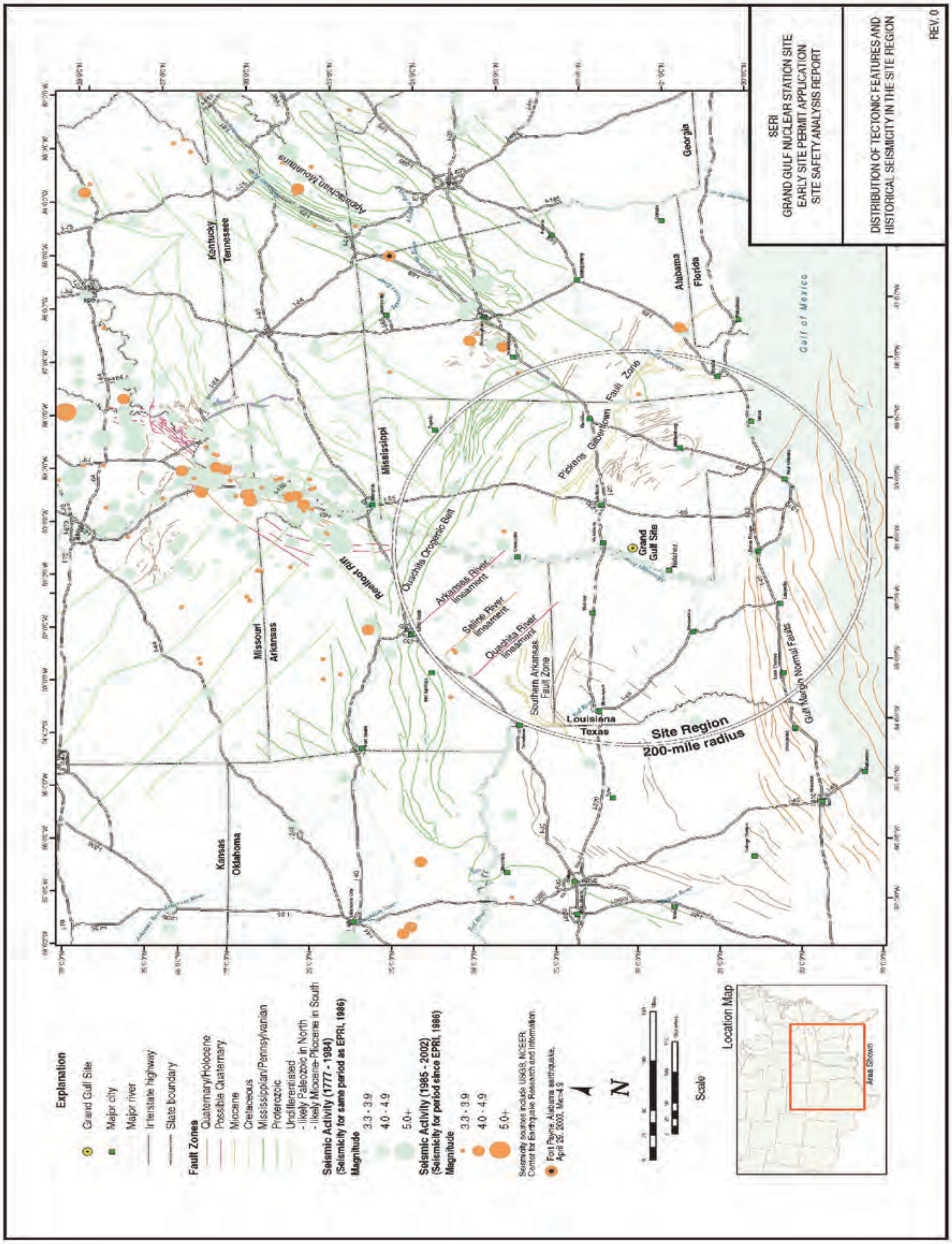


Figure 2.5.1-4 Tectonic features and seismicity in the site region

SSAR Sections 2.5.1.1.5.2 to 2.5.1.1.5.10 discuss each of the following seismic source zones surrounding the ESP site:

- Appalachian Mountains
- Ouachita Orogenic Belt
- Arkoma and Black Warrior Basins
- Reelfoot Rift
- New Madrid Seismic Zone (NMSZ)
- Gulf Coast Basin
- Pickens-Gilberttown and Southern Arkansas Fault Zones
- Saline River Source Zone (SRSZ)
- nontectonic structural features

Among these seismic sources, the SRSZ is a new addition to the original 1986 EPRI seismic hazard study, and the applicant's characterization of the NMSZ includes updated source parameters. Rather than characterizing the seismic potential of each specific geologic fault, the applicant used the areal seismic sources developed by the EPRI study, which are based on current seismicity. Descriptions of each of these seismic source zones follow, and Table 2.5.1-1 provides specific magnitude and historical earthquake parameters.

Table 2.5.1-1 Parameters for Seismic Source Zones in the Site Region

Tectonic Zones	Historical Earthquake (1777–1986)	Earthquake (1986–2002) m_b < 3.9	EPRI Maximum Earthquake Magnitude (Mw)
Appalachian Mountains	9	1	5.4–7.2
Ouachita Orogenic Belt	18	3	5.1–7.5
Arkoma and Black Warrior Basins	2	0	5.1–7.5
Reelfoot Rift	-	-	5.0–7.5
New Madrid Seismic Zone	-	-	7.1–7.9
Gulf Coast Basin	-	-	4.2–7.5
Pickens-Gilberttown and Southern Arkansas Fault Zones	6	1	4.2–7.5
Saline River Source Zone	9	3	6.0–7.0

Appalachian Mountains

The Appalachian Mountains extend from Newfoundland, Canada, to central Alabama. The Appalachian Mountains consist of a southwest-trending complex of folded, thrust, and metamorphosed terrains that developed over a period of approximately 800 million years. The mountains are approximately 2000 miles long and 400 miles wide. Structures of the Appalachian Mountains extend into the northeastern subsurface of the site region. The distance between the southern end of the Appalachian Mountains and the site is about 150 miles. Many Paleozoic thrust faults of regional extent exist within the Appalachian Mountains, but none of these faults have geological evidence of Quaternary activity. No distinct faults are identified as individual seismic sources within the Appalachian Mountains in the site region. Within the site region, only nine earthquakes with m_b 3.3–3.9 have occurred from

1777–1986. One earthquake with m_b less than 3.9 occurred within the subsurface extent of the Appalachian Mountains beneath the site region during the period from 1986–2002. Another earthquake of moment magnitude (Mw) 4.9 occurred in April 2003 within the Appalachian Mountains, outside the site region. The event occurred at a depth of 3 miles with a strike-slip mechanism and did not trigger any monitoring instrument at GGNS. Table 2.5.1-1 summarizes the historical seismic activity and relevant modeling parameters of the Appalachian Mountains.

Ouachita Orogenic Belt

The Ouachita Orogenic Belt is the eroded core of a mountain belt that formed as a result of an episode of continental collision and formation of the Pangea supercontinent during the Paleozoic Era. The Ouachita Orogenic Belt consists of complexly folded, thrust-faulted, and metamorphosed rocks, including accreted oceanic crust of Proterozoic age. The belt is approximately 1260 miles long and 50 miles wide, and 80 percent of its length is buried underneath Mesozoic and Tertiary sediments of the Gulf Coast Basin. Inside the site region, the southeastern Ouachita Orogenic Belt lies underneath the subsurface of northern Mississippi and southwestern Alabama. The closest distance between the southeastern end of the belt and the site is about 80 miles. The belt defines the northern edge of the Gulf Coast Basin, the southern margin of the Mississippi embayment, and the southern edge of the North American craton. The Ouachita Orogenic Belt was tectonically active until the late Paleozoic Era. The orogenic belt is in contact with a major decollement, along which marine sedimentary rocks from other plates thrust northward over the North American cratonic rocks. None of the large regional Paleozoic thrust faults inside the Ouachita Orogenic Belt display geological evidence of Quaternary activity, except some potential Quaternary active faults located in the southern part of the belt. Table 2.5.1-1 summarizes the historical seismic activity and relevant modeling parameters of the Ouachita Orogenic Belt.

Arkoma and Black Warrior Basins

The Arkoma and Black Warrior basins are located near the northern margin of the site region. Both basins contain sedimentary rocks associated with the Ouachita Orogenic Belt. The closest distance between the Arkoma and Black Warrior Basins and the site is about 260 kilometers. The major thrusting deformation for the basins ceased in the late Paleozoic to the early Mesozoic time period. The applicant did not identify any active tectonic features within the Arkoma and Black Warrior Basins, based on its review of previous research results. However, a swarm of small earthquakes with magnitudes less than 4.5 occurred in central Arkansas within the Arkoma Basin, just outside of the site region. Research results cited by the applicant indicate that this Enola earthquake swarm correlated spatially with a 1.6-mile-long, west-northwest-trending fault segment, relating to a basement listric fault. Favorable orientation between the basement listric fault and the current compressive stress may have caused the earthquake swarm. However, the applicant stated that large earthquakes will probably not occur in the area because two groups of faults intersect each other within the area (Schweig, E.S., R.B. Van Arsdale, and R.K. Burroughs, "Subsurface Structure in the Vicinity of an Intraplate Earthquake Swarm," 1991). Table 2.5.1-1 summarizes the historical seismic activity and relevant modeling parameters of the Arkoma and Black Warrior Basins.

Reelfoot Rift

The Reelfoot Rift represents a northeast-trending fault system that originated in Precambrian or early Cambrian time during the extension of the North American continent. The rift extends from southern Illinois at the northern end of the Mississippi embayment to east-central Arkansas and northern Mississippi. The Reelfoot Rift is approximately 45 miles wide and 180 miles long with as much as 25,000 feet of structural relief. The closest distance of the faults within the Reelfoot Rift and the site is approximately 175 miles. Magnetic anomalies that were caused by intrusive rocks define the boundaries of the rift. The rift contains a number of active tectonic features, including its own two margins, the Commerce Geophysical Lineament and the NMSZ, the primary seismic active area near the site region. The rift experienced numerous uplifts, subsidence, magma intrusions, and sedimentations. The seismic activity within the Reelfoot Rift is diffuse. Table 2.5.1-1 summarizes the historical seismic activity and EPRI modeling parameters for the Reelfoot Rift.

New Madrid Seismic Zone

The NMSZ extends from southeastern Missouri to northeastern Arkansas and northwestern Tennessee. The NMSZ is located within the Reelfoot Rift, and it experienced post-Eocene to Quaternary faulting and historical seismicity. The closest approach of the faults within the NMSZ to the site is approximately 260 miles. Although the NMSZ is outside the site region, it contributes significantly to the seismic hazard of the site. Rather than characterize the seismic potential of the known faults within the NMSZ, EPRI defines the NMSZ as an aerial source zone that is approximately 124 miles long and 25 miles wide. The NMSZ consists of three major fault segments—a southern, northeast-trending dextral slip fault, referred to as the Cottonwood Grove fault and Blytheville Arch; a middle, northwest-trending reverse fault, referred to as the Reelfoot fault; and a northern, northeast-trending dextral strike-slip fault, referred to as the East Prairie fault. Three large earthquakes occurred in 1811 and 1812 on each of these three faults. The estimated magnitudes of these earthquakes range from Mw 7.1–8.4, based on regional reports of damage intensity and the distribution of liquefaction features. The 1986 EPRI model defines magnitude ranges from Mw 7.3–8.7. The applicant stated that dates of paleoliquefaction and cross-cutting geological features suggest that the recurrence interval of a 1811–1812-type earthquake is 200–800 years, with a preferred estimate of 500 years. This recurrence estimate is significantly shorter than the 5000 years determined in the 1986 EPRI study. Table 2.5.1-1 summarizes the historical seismic activity and EPRI modeling parameters for the NMSZ.

Gulf Coast Basin

The Gulf Coast Basin is a north-south-trending syncline, approximately 280–400 miles wide, extending from eastern Texas to western Alabama and Florida and from southern Arkansas to the Gulf Coast. Bounded in the north by the Ouachita Orogenic Belt and in the east by the Appalachian Mountains, the Gulf Coast Basin defines a deep depression that contains more than 50,000 feet of Mesozoic and Cenozoic sediments. The ESP site is located inside the Gulf Coast Basin. Since post-Jurassic continental rifting and formation of the Gulf of Mexico, sedimentation is the dominant process for the basin. The amount of sediments transported to the Gulf Coast Basin exceeded the volume that could be accommodated through basin subsidence and infilling; therefore, the sedimentary complex migrated southward over 250 miles. Development of a series of growth faults, defining the margins of unstable shelves,

marked each depocenter shifting. Current growth faults are located along the Cretaceous shelf edge in the vicinity of the modern Gulf Coast, 90 miles south of the ESP site. Table 2.5.1-1 summarizes the historical seismic activity and EPRI modeling parameters for the Gulf Coast Basin.

Pickens-Gilberttown and Southern Arkansas Fault Zones

The Pickens-Gilberttown and Southern Arkansas Fault Zones are a system of faults extending from southwestern Alabama though west-central Mississippi to southern Arkansas and eastern Texas. The Pickens-Gilberttown and Southern Arkansas Fault Zones consist of a series of grabens developed in Paleozoic to middle Tertiary deposits on the southward side of the Ouachita Orogenic Belt. The fault zones are more than 500 miles long and 25 miles wide. The closest approach of faults within the Pickens-Gilberttown and Southern Arkansas Fault Zones to the site is approximately 70 miles. The fault zones offset Miocene sediments by as much as 200 feet and pre-Miocene deposits by as much as 1000 feet. However, the applicant stated that the Pliocene and younger deposits are intact, which indicates that the fault zone has not been active since the Miocene Epoch. The Pickens-Gilberttown and Southern Arkansas Fault Zones formed through gravitational collapse caused by the uneven loads in the Tertiary Gulf Coast Plain, analogues to the currently active gulf margin normal faults. Very little seismicity exists along the Pickens-Gilberttown and Southern Arkansas Fault Zones. Table 2.5.1-1 summarizes the historical seismic activity and EPRI modeling parameters for the fault zones.

Saline River Source Zone

The SRSZ is located in southeastern Arkansas and northwestern Mississippi, with a minor extension into northern Louisiana. The SRSZ lies inside the Ouachita Orogenic Belt and structurally overlies the southwestward subsurface extension of the Proterozoic Reelfoot Rift. The closest approach of faults within the SRSZ to the site is approximately 175 miles. The applicant stated that Cox (Cox, R.T., "Investigation of Seismically-Induced Liquefaction in the Southern Mississippi Embayment," National Earthquake Hazards Reduction Program, Final Technical Report No. 01HQGR0052, U.S. Geological Survey, Reston, Virginia, 2002) identifies the SRSZ based on combined, although nonconclusive, evidence from geomorphology, geology, and paleoseismology. Geomorphic evidence includes asymmetry of drainage basins and the relative locations of terraces probably caused by the southwestward migration of the Ouachita, Saline, and Arkansas Rivers. Trenching and road-cut exposures show possible post-Eocene and Quaternary faulting. The vertical slip rates estimated from the incision of streams into terraces range from 0.05–1.7 millimeters per year (mm/yr). Paleoliquefaction evidence includes sand dikes found at three sites in Ashley County and Kelso in Desha County. The applicant established an earthquake chronology for the area based on liquefaction events identified through stratigraphic and cross-cutting relationships, as well as datable materials found in those sites. A four-event scenario resulted in a return period of 1725 years, and a five-event scenario resulted in a return period of 388 years for the seismic zone. The difference between minimum and maximum dated events produces an upbound recurrence interval of 3500 years. The estimated slip rates for the SRSZ range from 0.05–1.7 mm/yr based on estimates of incision rates, and range from 0.008–0.03 mm/yr based on measured fault displacement.

According to the applicant, if local events induced the liquefaction distribution, the estimated maximum magnitude for the SRSZ ranges from 5.5–6.0 based on the area of liquefaction distribution. However, if the New Madrid seismic events caused the liquefaction, the SRSZ will

lack supporting grounds because the evidence from geology and geomorphology is not conclusive and may alternatively be explained by activity along the Reelfoot Rift or even nontectonic processes. Table 2.5.1-1 summarizes the historical seismic activity and EPRI modeling parameters for the SRSZ.

The applicant cited the data extensively from Cox in describing the SRSZ. In RAI 2.5.1-3, the NRC staff asked the applicant to explain the degree to which the latest findings of Cox and others (Cox, R.T., Larsen, D., Forman, S.L., Woods, J., Morat, J., and Galluzzi, J., "Preliminary Assessment of Sand Blows in the Southern Mississippi Embayment," Bulletin of the Seismological Society of America, 2004) are consistent with the applicant's characterization of the SRSZ. In response, the applicant stated that it established the earthquake chronology and recurrence based on the observed stratigraphic relationships reported in Cox and associated radiocarbon dates provided by Cox and finally published by Cox and others (Cox, et al., 2004). The applicant also described that it used two standard deviations, instead of the single standard deviation employed by Cox, to account for the full range of uncertainty. Further, according to the applicant, Cox did not present in his publications a detailed event history that can be used to assess the recurrence intervals. The staff also asked the applicant in RAI 2.5.1-3 to explain whether the large sizes of the sand blows and their positioning at or close to the edge of their liquefaction distribution are consistent with the moderate magnitudes estimates. In its response, the applicant stated that the lack of geotechnical data at the SRSZ does not allow for a complete analysis to estimate the magnitudes for the paleoearthquakes, but the area of the liquefaction distributions are consistent with the moderate earthquake estimated for the source zone.

The staff also asked the applicant in RAI 2.5.1-3 to explain the source of SSAR Table 2.5-5 and provide the reasoning for the values listed in Tables 2.5-5 and 2.5-6. The staff specifically asked the applicant to provide a link between the events listed in SSAR Section 2.5.1.1.5.9.3 and Table 2.5-5. In its response, the applicant stated that the chronologic liquefaction events provided in Table 2.5-5 are based on the summary of all the identified events dated by carbon-14 and other dependable methods. Among them, one to three possible events occurred at the Portland site, two to four events took place at the Montrose site, and three events happened at the Kelso site. The applicant explained that it estimated the average recurrence intervals based on the possible number of liquefaction events and the minimum and maximum allowable time in which these events could occur. The interpretation assumes that all of the liquefaction features could have occurred as a result of four or five local events in the SRSZ. The applicant revised SSAR Section 2.5.1 as the result of this RAI and added four new figures to further illustrate the process to establish an earthquake chronology for the area. Finally, the staff asked the applicant to explain the availability of the paleoliquefaction dates from the Morgan and Golden trenches with respect to the finishing date of the SSAR and the impact of the new paleoliquefaction dates on the conclusions in the SSAR. In response, the applicant stated that it reviewed new data from the Golden and Morgan sites after they became available. It found that the precision of age dates for the Golden and Morgan sites is not sufficient to date specific liquefaction events, although the interpreted paleoliquefaction events are consistent with the mid-Holocene events identified at the Montrose and Kelso sites.

In RAI 2.5.1-4, the staff asked the applicant to explain the impact of the remote source scenario that attributes liquefaction found in the Saline River area near the NMSZ to the magnitude determination of the NMSZ. In its response, the applicant stated that, based on an empirical relationship (Ambraseys, N.N., "Engineering Seismology: Earthquake Engineering and Structural Dynamics," Elsevier Science, 1988) between liquefaction distribution and magnitude,

the southern Blytheville Arch segment, with an estimated magnitude range of Mw 7.3–8.1, could produce liquefaction 112–218 miles away from the closest approach of the fault. In addition, the estimated distances should be viewed as minimum estimates because the Ambrasey's relationship (Ambraseys, 1988) is based on a worldwide dataset that includes a majority of plate boundary events with high attenuation rates, while the CEUS is characterized by much lower attenuation rates. Therefore, the Ambrasey's relationship can account for the liquefaction found in southeastern Arkansas.

Nontectonic Structural Features

Nontectonic structural features in the site region and neighboring area include volcanic domes, salt domes, and growth faults. These nontectonic features deformed the sediments at some locations inside the Gulf Coast Basin, where the ESP site is located. The seismic source modeling process for the ESP site did not include any of these nontectonic features.

Volcanic domes are the main nontectonic features in or near the site region. The two most prominent volcanic domes located in the site region are the Jackson Dome and Monroe Uplift. The Jackson Dome is a circular volcanic plug with a 16-mile diameter, located at the southern margin of the Mississippi embayment near the city of Jackson. The center of the dome is about 62 miles away from the site. The dome became active in the early Cretaceous Period, continued to rise through post-Oligocene time, and has a total structural relief of about 10,000 feet. Radiometric dating shows a possible long-term quiescence in its active history. Seismic line interpretation reveals several faults associated with the dome (Dockery, D.T., III, and Marble, J.C., "Seismic Stratigraphy of the Jackton Dome," Mississippi Geology, 1998). The Monroe Uplift is a volcanic dome that straddles southern Arkansas, northern Louisiana, and west-central Mississippi. The center of the uplift is about 75 miles away from the site, and the uplift is characterized by the arching of strata above a deep-seated igneous intrusion. The Monroe Uplift became active in the Jurassic Period and experienced continued movement into post-Miocene time. The uplift has no surface expression.

Salt domes are located in the Interior Salt Basin and the Coastal Salt Basin within the Gulf Coastal Plain. The source of the salt is the Middle Jurassic Louann Salt in the Interior Salt Basin and the Coastal Salt Basin. Salt migration structures are concentrated in a zone approximately 156 miles wide extending from southwestern Alabama to eastern Texas. The migration of salt produced anticlines, diapiric folds, and piercement domes. The source depth for the Louann Salt in this area is approximately 15,000 feet and becomes progressively deeper to the south. The closest salt dome, the Bruinsburg Dome, is approximately 6.5 miles from the ESP site. Salt domes within the Interior Salt Basin have not been active since the Oligocene Epoch. Salt domes in the Coastal Salt Basin formed in the Miocene Epoch have been active through the Quaternary Period and deform the ground surface.

Gulf margin normal faults, which are located in the southern margin of the Gulf Coast Basin, include the Tepetate-Baton Rouge, Denham Springs-Scotlandville, and Lake Hatch faults, as well as many unnamed faults. These normal faults generally trend east-west, and the closest fault is approximately 90 miles from the ESP site. The Louann Salt forms a sliding layer on which the overlying sedimentary section has mobilized, forming a series of Tertiary and Quaternary growth faults. These faults associated with Louann Salt generally dip between 50 and 70 degrees at the surface and less than 50 degrees at depth. The current gulf margin

normal faults are located along the subsurface Cretaceous shelf edge and experience high rates of aseismic slip.

Regional Seismicity. The applicant stated that historic seismicity in the region is mostly concentrated in the Reelfoot Rift and the NMSZ, which are underlain by crystalline rocks of the North American craton. Small-magnitude earthquakes also occur along the general trend of the Ouachita Orogenic Belt and Appalachian Mountains. The historic seismic rate is very low inside the site region. Since 1777, records indicate 1 earthquake of approximately m_b 3.3–3.9 within 90 miles and 39 earthquakes of m_b greater than 3.3 within 200 miles of the site. No earthquakes were ever recorded in the site vicinity during the same period.

In Sections 2.5.1 and 2.5.2 of the SSAR, the applicant described its use of an earthquake catalog with a magnitude cutoff of m_b 3.3. In RAI 2.5.1-1, the staff asked the applicant to explain whether this magnitude cutoff of m_b 3.3 resulted in excluding seismicity along the Gulf Coast of Louisiana and a 1927 earthquake of m_b 3.0 west of Jackson, Mississippi. In addition, the staff asked the applicant to explain the absence of an earthquake of m_b 3.4 in Jackson, Mississippi. As a result of the RAI, the applicant revised the regional seismic catalog with a lower cutoff magnitude of m_b 1.0 and modified the appropriate text and replaced the relevant tables and figures in the SSAR.

In RAI 2.5.1-2, the staff asked the applicant to provide scientific evidence for its statement, “This low rate of activity has characterized the seismicity of the Gulf Plain for over 150 years, and most likely throughout the Quaternary.” In its response, the applicant stated the following:

> The statement was intended to highlight the fact that rates of both earthquake occurrence and tectonic deformation in the site region are extremely low. The Gulf Coast Plain has been characterized by extremely low rates of tectonic deformation with post-Cretaceous deposits on the limbs of the Gulf Coast Syncline dipping less that 0.5 degrees. Based on the low rates of tectonic deformation that have occurred over a period of tens of millions of years and the low rates of seismic activity over the last 150 years, we infer that these rates were most likely also characteristic of Quaternary period.

2.5.1.1.2 Site Geology

Section 2.5.1.2 of the ESP application describes the geologic information of both the site area (8 kilometers) and the site location (1 kilometer) in terms of the (1) site physiography and geomorphology, (2) site geologic history, (3) site geologic conditions, (4) site structure, and (5) geotechnical properties of subsurface materials.

Site Physiography and Geomorphology. SSAR Section 2.5.1.2.1 describes the site physiography and geomorphology of the site area, including the site location. According to the applicant, the ESP site is approximately 1.1 miles east of the Mississippi River and adjacent to the Mississippi River floodplain. The boundary between the Mississippi Alluvial Valley and Loess Hills physiographic subprovinces, marked by a 65–80-foot-high, north-trending erosional escarpment at the edge of the Mississippi River floodplain, strides the site location. Steep-walled stream valleys, flat-topped ridgelines, and dendritic (tree-patterned) drainage systems are the main topographic features of the Loess Hills subprovince. Large river terraces

occur along the river floodplains and valley bottoms. Floodplain, cut-banks, point bars, and oxbow lakes characterize the surface of the Mississippi River Valley, and the topography of the valley is relatively flat. The applicant further stated that the radius of the site location does not extend to the active channel of the Mississippi River. The proposed facility location is bounded on the east by existing internal plant roads and parking lots, on the west by the erosional escarpment at the edge of the Mississippi River floodplain, and on the north and south by two ravines that drain the location.

Site Geologic History. SSAR Section 2.5.1.2.2 describes the geologic history of the site area, including the site location. According to the applicant, the geologic formations underlying the site area and site location include both marine and terrestrial sediments that reflect distinct changes in depositional environments, climatic conditions, and glacial-eustatic cycles over the past 36 million years. Deposits of at least Oligocene age (37–24 Ma) and younger dip very gently southward and are laterally continuous across the site region. The applicant concluded that these deposits are relatively intact and thus document a long-term tectonic stability.

The Oligocene deposits in the site area, represented by the Glendon Limestone and Byram Marl Formations of the Vicksburg Group, reflect a shallow marine seas environment. The Glendon Limestone occurs at a depth of approximately 300 feet beneath the site. The Byram Marl Formations were overlain by the late Oligocene Bucatunna Clay Formation, indicating a transition to a deep-water or estuarine environment. These Oligocene sedimentary deposits are in contact unconformably with the Miocene (24–5 Ma) Catahoula formation, which consists mainly of silty to sandy clay, clayey silts, and sands. The Catahoula formation represents a marginal shoreline depositing environment. Because coarse sands and gravels of the Pliocene and Pleistocene Epochs unconformably overlie the top of the Catahoula formation, the applicant determined that the marginal shoreline environment migrated into an alluvial environment during the Pliocene (5–1.8 Ma) and Pleistocene (1.8–0.11 Ma) Epochs. Wisconsin-age glacial cycles that supplied a large volume of sediments to the Mississippi Alluvial Valley led to the deposition of sediments of the late Pleistocene terraces, Upland Complex. During the late Pleistocene Epoch, the main deposits in the area were loess deposits, which consist mainly of fine-grained sediments transported by wind. The average thickness of the loess in the site location is approximately 65 feet. Tributary streams to the Mississippi River eroded the loess deposits during the Holocene Epoch. Meanwhile, the Mississippi River floodplain in the western part of the site area and site location also accumulated alluvial sediments. Deposition of alluvial deposits during peak glacial outwash may have changed local base levels of erosion, blocking stream outlets and leading to the ponding or deposition of silt and alluvium tributary valleys. The applicant further stated that the subsequent drop in the river level during the current interglacial period may have caused the incision and formation of the terrace remnants along Bayou Pierre and the Big Black River.

Site Geologic Conditions. SSAR Section 2.5.1.2.3 describes geologic conditions of the site area, including the site location. The applicant stated that the Quaternary (including both Holocene and Pleistocene) deposits in the site area and site location include gravels, sands, silts, clays, and loess formed by fluvial processes along the river system and eolian processes along the eastern margin of the Mississippi Alluvial Valley. Specifically, the Holocene fluvial deposits are located on the floodplain of the Mississippi River, the alluvium and terrace deposits are located in tributary valleys, and the colluvium is located along hill slopes in the Loess Hills. The thickness of the Holocene deposits in the site area ranges from 22–182 feet. Terrace and

loess deposits are two types of Pleistocene deposits in the site area and site location. Terrace deposits occur in the site area along the Loess Hill bluff, Bayou Pierre, and small tributary streams. The terraces occur at three elevation levels—140, 160, and 180 feet, respectively. The location of the prospective reactor lies on an inferred late-Pleistocene terrace surface at an elevation of approximately 150 feet. Beneath the terrace deposits lie the loess deposits with a thickness of up to 75 feet. Three Pleistocene-age loess sheets occur between Vicksburg and Natchez in the site vicinity. The loess deposits are up to 100-feet thick and consist of yellowish-brown, medium-stiff, sandy to clayey silt with a weak block structure. The loess deposit lies on top of the coarse-grained alluvial sand and gravel deposits of the Upland Complex.

Including the Upland Complex, all the Tertiary deposits do not reach to the surface in the site area. The youngest Tertiary deposits, the Upland Complex, consist of two alluvial layers beneath the proposed site. The upper alluvial deposit is located at an elevation of 68–71 feet with a thickness of about 46–85 feet and consists of light-gray to brownish-yellow sand and silty sand. The silty sand consists of fine- to medium-grained, well-sorted quartz grains with silt and is massive, dense, and friable. The lower alluvial deposit is located between elevations of 24–14 feet and ranges from 11–89 feet in thickness across the proposed site, which consists of stratified, thinly bedded sands, silty clays, and gravels. Immediately beneath the Upland Complex lies the Miocene Catahoula formation, a combination of hard to very hard silty to sandy clay, clayey silt, and sand. The formation occurs at depths of 125–175 feet in the site area. The Catahoula formation would serve as the load-bearing stratum for the nuclear plant structures at the ESP site. Beneath the Catahoula formation lies the Vicksburg Group, the upper part of the Oligocene deposits. The Vicksburg Group mainly consists of clay, marl, limestone, and calcareous clays.

Site Structures. SSAR Section 2.5.1.2.4 describes the structure of the site area, including the site location. The applicant focused on active faults and other structures in the site area but found no indication of tectonic deformation since the Oligocene period.

Faults and Unconformities

The applicant stated that field mapping found no faults within the 8-kilometer radius of the site area. The lateral continuity of subsurface stratigraphy demonstrates the tectonic stability of the site area and site vicinity from at least the Oligocene time, about 37 Ma to present. The top of the Glendon Limestone Formation of the Vicksburg Group shows no morphology indicative of tectonic deformation. The top of the Catahoula formation and the top of the Upland Complex also show no morphology indicative of tectonic deformation. No new information available since the licensee’s original investigations for GGNS suggests the presence of faulting within the site area. Most contacts among the subsurface deposits in the site area and site location are erosional unconformities, except for the conformable contact between the Oligocene Glendon Limestone and the Byram Marl Formation of the Vicksburg Group.

Other Structures

The site is located along the northern margin of the Mississippi Salt Basin. However, no salt domes occur in the site area or site location. The nearest salt dome is the Bruinsburg Dome located 6.5 miles southwest of the site. The depth to the salt of the Bruinsburg Dome is 2200 feet. The Bruinsburg Dome has unwarped the Glendon Limestone strata in the site vicinity but does not affect the Miocene Catahoula formation.

Conditions Caused by Human Activity

According to the applicant, no mining or underground mineral extraction activities are near the site and no petroleum producing area is within 10 miles of the site. Ground water extraction is nominal in the area; no zones of observed ground water level depression are present within 5 miles of the site. The applicant also noted that it did not expect significant future mineral and underground water extraction activity in the area.

2.5.1.2 Regulatory Evaluation

SSAR Section 2.5.1 reviews and summarizes the geological and seismological characteristics of the ESP site progressively from a regional scale to the local site. According to the applicant, its assessment addresses the requirements in 10 CFR Part 52. The applicant also stated that it performed the analyses in accordance with the requirements for development of the SSE ground motions provided in Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50 and conformed to the criteria set forth in 10 CFR 100.23. The applicant also performed its analyses following the guidelines in RG 1.165; RG 1.132, Revision 1, "Site Investigations for Foundations of Nuclear Power Plants," issued March 1979; and RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," dated April 30, 1978. To satisfy the requirement in 10 CFR 100.23, the applicant described the geological and seismological characteristics of the ESP site to allow an adequate evaluation of the proposed site, to provide sufficient information to support the SSE estimates, and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. The staff notes that GDC 2 applies to this portion of the review of an ESP application only with regard to consideration of the most severe natural phenomena reported for the site (in this case earthquake), including margin.

In reviewing the SSAR, the staff considered the regulations at 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23(c), which require that the applicant for an ESP describe the seismic and geologic characteristics of the proposed site. In particular, 10 CFR 100.23(c) requires that an ESP applicant investigate the geological, seismological, and engineering characteristics of the proposed site and its environs with sufficient scope and detail to support evaluations to estimate the SSE and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the site. Section 2.5.1 of NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," issued 1997, RG 1.165, and Section 2.5 of RG 1.70, Revision 3, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants–LWR Edition," November 1978, provide specific guidance concerning the evaluation of information characterizing the geology and seismology of a proposed site.

2.5.1.3 Technical Evaluation

This section of the SER provides the staff's evaluation of the geological and seismological information submitted by the applicant in SSAR Section 2.5.1. The technical information presented in Section 2.5.1 of the application resulted from the applicant's surface and subsurface geological and seismological investigations performed in progressively greater detail as these investigations approached the site. Through its review, the staff determined whether the applicant complied with the applicable regulations and conducted its investigations with an appropriate level of thoroughness, as required by 10 CFR 100.23. SSAR Section 2.5.1

contains the geologic and seismic information gathered by the applicant in support of the vibratory ground motion analysis and site SSE spectrum provided in SSAR Section 2.5.2. According to RG 1.165, applicants may develop the vibratory design ground motion for a new nuclear power plant using either the EPRI or LLNL seismic source models for the CEUS. However, RG 1.165 recommends that applicants update the geological, seismological, and geophysical database and evaluate any new data to determine whether revisions to the EPRI or LLNL seismic source models are necessary. As a result, the staff focused its review on geologic and seismic data published since the late 1980s that could indicate a need for changes to the EPRI or LLNL seismic source models.

To thoroughly evaluate the geological and seismological information presented by the applicant, the staff obtained the assistance of USGS. In addition, the staff and its USGS advisors visited the ESP site and surrounding area to evaluate and confirm the interpretations, assumptions, and conclusions presented by the applicant concerning potential geologic and seismic hazards. The staff's review focused on the applicant's characterization of the regional and local geologic structure and seismic potential.

2.5.1.3.1 Regional Geology

The staff focused its review of SSAR Section 2.5.1.1 on the applicant's description of the regional geologic and tectonic setting of the ESP site, with an emphasis on the neotectonics, seismology, paleoseismology, physiography, geomorphology, stratigraphy, and geologic history within a distance of 200 miles from the site.

In SSAR Section 2.5.1.1.1, the applicant described each of the physiographic provinces within the site region, with an emphasis on the Gulf Coast Plain where the ESP site is located. In SSAR Section 2.5.1.1.2, the applicant described the Mississippi embayment and Gulf Coast Basin, two macroscopic geologic features of the ESP site region. In SSAR Section 2.5.1.1.3, the applicant summarized the tectonic evolution starting from the Precambrian Era for major geologic features, focusing more on recent geologic activities. In SSAR Section 2.5.1.1.4, the applicant described the regional stratigraphy of the site region with emphasis on the younger stratigraphy and the major rock units underlying the site. These four SSAR sections describe well-documented geologic information, and the staff concludes that they contain an accurate and thorough description of the regional geology as required by 10 CFR 52.17 and 10 CFR 100.23.

In SSAR Section 2.5.1.1.5, the applicant described the tectonic features included in the EPRI seismic source model from the late 1980s, focusing on the NMSZ and the SRSZ. This model was either updated or included as a new addition to the original EPRI seismic source model. The applicant compared the parameters used in the original EPRI model and parameters obtained from the latest studies and correlated these tectonic features with seismic activities in the region. The applicant summarized the latest geological and seismological studies for the NMSZ, especially the paleoseismological studies that provide the new geometry and estimates on earthquake magnitudes and recurrence intervals for the seismic zone. The applicant described in detail the uncovered geologic, geomorphologic, and paleoseismologic evidence and existing interpretations of the evidence for the SRSZ. In responding to RAI 2.5.1-3, the applicant stated that it used data from Cox and others (Cox, et al., 2004), but expanded from one to two standard deviations to account for the full range of uncertainty in the dates of the

paleoliquefaction events. Based on these dates, the applicant established the multiscenario earthquake chronology for the SRSZ. Although the evidence of the SRSZ is not conclusive and the interpretations of the paleoliquefaction events are subjective and ambiguous, the addition of this particular seismic source to the ESP seismic hazard calculation only enhances the conservative estimate of ground motions for the ESP site. With this consideration, the staff concurs with the applicant's conclusions about the characterization of the SRSZ. In responding to RAI 2.5.1-4, the applicant explained that existing magnitude estimates of the NMSZ can account for the liquefaction found in southeastern Arkansas, based on the Ambrasey's relationship (Ambrasey, 1988) between liquefaction distribution and magnitude. The staff concurs with the applicant's conclusion that the liquefaction found in southeastern Arkansas can be accounted for by the existing magnitude estimates of the NMSZ. In reviewing SSAR Section 2.5.1.1.5 and the applicant's responses to the RAIs, the staff concludes that the applicant accurately characterized the tectonic features and their correlations with the regional seismicity, as required by 10 CFR 52.17 and 10 CFR 100.23.

In SSAR Section 2.5.1.1.6, the applicant described the regional seismicity. The applicant concluded that limited earthquake occurrence since 1777 characterizes the ESP site region as a low seismic-activity region, and most earthquake activities are concentrated along the reactivated older tectonic features, such as the Reelfoot Rift zone. Moreover, no faults are mapped within a 90-mile radius, centered at the ESP site. In responding to RAI 2.5.1-1, the applicant explained that it used m_b 3.3 as the magnitude cutoff for the regional earthquake catalog because it is the same cutoff employed in the earthquake catalog in the original EPRI model. As a result of this RAI, the applicant presented a new seismic catalog with a magnitude cutoff of m_b 1.0. Section 2.5.1.1.6 of the SSAR summarizes the applicant's revisions resulting from this RAI. The applicant also responded to RAI 2.5.1-2 by explaining that the observation of no tectonic deformation in the post-Cretaceous deposits in the site area, as well as 150 years of low seismic activity, led it to infer that the low rates were mostly characteristic of the entire Quaternary Period. The applicant indicated that the statement is qualitative in nature and does not affect the assessment of seismic hazard at the ESP site. In reviewing the applicant's response, the staff concludes that this RAI is resolved. Based on its review of SSAR 2.5.1.1.6, in addition to the applicant's responses to the RAIs cited above, the staff concludes that SSAR Section 2.5.1.1.6 provides an accurate and thorough description of the regional seismicity, as required by 10 CFR 52.17 and 10 CFR 100.23.

2.5.1.3.2 Site Geology

The applicant presented surface and subsurface geologic information covering both the site area and site location in SSAR Section 2.5.1.2. In SSAR Section 2.5.1.2.1, the applicant described in detail the spatial relationship between the ESP site and the relevant physiographic provinces inside the site area. In SSAR Section 2.5.1.2.2, the applicant described the site area's geologic evolution since the late Tertiary Period (Oligocene Epoch). The applicant concluded that the geologic formations underlying the area indicate a long history of tectonic stability and the absence of tectonic deformation. Based on its review, the staff concludes that the applicant provided a thorough and accurate description of the surface features and characteristics for the ESP site in support of the ESP application in these two sections.

In SSAR Section 2.5.1.2.3, the applicant described major stratigraphic units since the Oligocene Epoch based on previous and present subsurface investigations. In particular, the applicant characterized the load-bearing stratum for the existing nuclear facility and the prospective facility. In SSAR Section 2.5.1.2.4, the applicant described the geological

structures and other structures in the site area based on the analysis of the surface and subsurface layer continuity. The staff concludes, based on its review, that the applicant's SSAR provides an accurate and thorough description of the site area stratigraphy, with emphasis on the younger layers of rock and soils. The staff also concludes that the applicant's description of the geological structures is complete and accurate. Based on RG 1.132, excavation made during construction provides opportunities for obtaining additional geologic and geotechnical data. Therefore, it is necessary to perform geologic mapping of future excavation for safety-related structures, evaluate any unforseen geologic features that are encountered, and notify the NRC no later than 30 days before any excavations for safety-related structures are open for NRC's examination and evaluation. This is **Permit Condition 3.**

In SSAR Section 2.5.1.2.5, the applicant described possible supporting materials for the proposed nuclear facility foundation. After reviewing, the staff concludes that this very brief section only addressed the possible depths and foundation layers for the potential reactor and the applicant's detailed description about subsurface materials is in SSAR Section 2.5.4.

Finally, the applicant discussed potential hazard conditions caused by human activities, such as ground water depression and ground surface stability related to mining activities at the site. Based on its review, the staff agrees with the applicant's assessments and concludes that no potential exists for hazard conditions, such as subsidence or collapse, caused by human activity that could compromise the safety of the site.

In summary, the staff concludes that the contents presented in these six sections regarding the site geology meet the requirements in 10 CFR 52.17 and 10 CFR 100.23.

2.5.1.4 Conclusions

As set forth above, the staff has reviewed the geological and seismological information submitted by the applicant in SSAR Section 2.5.1. On the basis of its review and as described above, the staff finds that the applicant provided a thorough characterization of the geological and seismological characteristics of the site, as required by 10 CFR 100.23. These results provide an adequate review for all the tectonic sources or seismogenic sources that have the potential for seismic impact to the ESP site, either inside the site region or outside of it, but still nearby. In addition, the staff concludes, as described above, that the applicant has identified and appropriately characterized all the seismic sources significant for determining the SSE for the ESP site, in accordance with RG 1.165 and Section 2.5.1 of NUREG-0800, and therefore satisfied, in this respect, the requirements of 10 CFR 100.23(c) and GDC 2. Based on the applicant's geological, geophysical, and geotechnical investigations of the site vicinity and site area, the staff concludes that the applicant has properly characterized the site lithology, stratigraphy, geological history, structural geology, and the characteristics of subsurface soils and rocks. Based on its review of the material presented in SSAR Section 2.5.1, the staff also concludes that the effects of human activity (e.g., ground water withdrawal or mining activity) have no potential to compromise the safety of the site. Therefore, the staff concludes that the proposed ESP site is acceptable from a geological and seismological standpoint and meets the requirements of 10 CFR 100.23.

2.5.2 Vibratory Ground Motion

SSAR Section 2.5.1 describes the regional and local geology and structural background and outlines the major seismotectonic sources and materials in the site region. Based on the

background knowledge of the area, SSAR Section 2.5.2 describes the applicant's determination of the ground motions at the ESP site resulting from possible earthquakes inside or even outside the site region. SSAR Section 2.5.2.1 describes the characteristics of seismic sources used in the ESP site seismic hazard calculation. Section 2.5.2.2 presents the procedure for the probabilistic seismic hazard analysis (PSHA) and its results. Sections 2.5.2.3 and 2.5.2.4 of the SSAR describe site characteristics in seismic wave transmission and site responses at the ESP site. Finally, SSAR Sections 2.5.2.5 and 2.5.2.6 summarize the development of the SSE and operating-basis earthquake (OBE) ground motion for the ESP site.

The applicant stated that the information provided in SSAR Section 2.5.2 complies with the procedure recommended in RG 1.165. The four-step procedure includes (1) reviewing the EPRI and LLNL seismic source model and ground motion model, (2) updating these models with new information, (3) performing a new PSHA based on previous updates, and (4) developing the SSE using those updated results with consideration of the site characteristics. In particular, the applicant stated that it adopted the 1986 EPRI-Seismicity Owners Group (SOG) methodology including the seismic source model developed by six earth science teams (ESTs). Based on its review of the latest research, the applicant also added the new SRSZ and the characteristic earthquake model for the NMSZ over the original EPRI seismic source model. In addition, the applicant computed seismic ground motion using the EPRI-SOG new ground motion model (EPRI 1008910, "EUS Ground Motion Project-Model Development and Results," issued August 2003) for the CEUS. Finally, the applicant conducted a site-specific site response analysis to develop the SSE at the ESP site.

2.5.2.1 Technical Information in the Application

2.5.2.1.1 Seismic Source Characterization

SSAR Section 2.5.2.1 describes the characteristics of all seismic sources in the ESP site region. In Section 2.5.2.1.1, the applicant reviewed the original 1986 EPRI earthquake source model related to the ESP site and found that the model adequately captures the regional earthquake source characteristics and the uncertainty associated with the source model at the time when the model was developed. The applicant addressed two new seismic sources and their associated parameters resulting from the recent studies described in SSAR Sections 2.5.2.1.2 and 2.5.2.1.3, respectively.

<u>Summary of EPRI Seismic Source Model</u>. SSAR Section 2.5.2.1.1 summarizes the original 1986 EPRI-SOG source model and parameters. The applicant stated that six independent ESTs are involved in characterizing CEUS seismic sources in the EPRI project. These ESTs evaluated geological, geophysical, and seismological data to model the occurrence of future earthquakes and evaluate earthquake hazards at nuclear power plant sites in the CEUS. The six ESTs involved in the EPRI project included (1) the Bechtel Group, (2) Dames and Moore, (3) Law Engineering, (4) Roundout Associates, (5) Weston Geophysical Corporation, and (6) Woodward-Clyde Consultants. In 1989, EPRI implemented the results of the seismic source characterizations with modification and simplification from each of the ESTs in a PSHA for nuclear power plant sites in the CEUS. The applicant stated that the parameters used in the 1989 PSHA calculations are the primary source for the seismic parameters used in this ESP calculation. SSAR Tables 2.5-8a through 2.5-8f summarize the seismic source information developed by each of the ESTs for seismic sources in the site region, and SSAR Figures 2.5-39 through 2.5-44 show the geometry of these seismic sources. This source information includes the maximum magnitude, closest distance to the ESP site, probability of activity, and an

indication as to whether new information regarding the seismic source has been identified since the original EPRI seismic hazard analyses. SSAR Section 2.5.2 does not present earthquake recurrence values for each of the seismic sources because they were computed for each 1-degree latitude and longitude cell that intersects any portion of a seismic source.

In RAI 2.5.2-6, the staff asked the applicant to provide a justification for not updating the EPRI 1986 seismic source characterizations to give more weight to larger magnitude earthquakes for the seismic source surrounding the site, following the 1994 Johnston studies (Johnston, A.C., et al., TR-102261-VI, "The Earthquakes of Stable Continental Regions," 1994). In its response, the applicant stated that the evaluation process of seismic sources by the ESTs is equivalent to a Level 4 analysis recommended by the Senior Seismic Hazard Advisory Committee (1997). The applicant did not find any changes in maximum magnitude, seismicity distribution, or rate of occurrence in reviewing earthquake activity after 1986. In addition, the applicant stated that in the site vicinity, no known Mesozoic and younger tectonic structures exist that may be reactivated, and the site region is underlain by relatively undeformed Cretaceous and younger strata over 10,000 feet thick. The applicant further explained that Johnston's preliminary research results were available to the ESTs and that the individual teams even adopted Johnston's estimate for the maximum magnitude for the Gulf Coastal region. The applicant believed that the uncertainty bounds of the EPRI seismic model encompass the maximum magnitude proposed by the studies of Johnston et al. Based on the information presented, the applicant concluded that Johnston's final results on the background seismic source for the ESP site do not provide new information that would significantly change the maximum magnitude estimates, probability of occurrence, recurrence models, and source geometry of the corresponding source inside the EPRI 1986 seismic source model.

<u>Characterization of the New Madrid Seismic Zone</u>. SSAR Section 2.5.2.1.2 describes the latest seismic and paleoseismic studies and the new parameters of the characteristic model of the NMSZ. The applicant stated that it overlapped the characteristic NMSZ seismic model on the EPRI original aerial source model in the PSHA for the ESP site. A three-fault configuration (the Blytheville Arch fault (New Madrid South), the Reelfoot fault, and the East Prairie fault (New Madrid North)) represents the characteristic NMSZ model. The applicant modeled the NMSZ earthquakes using a point-source model, assuming that the earthquakes occur on the southernmost end of each fault because of the large source-to-site distance. The applicant also noted that dissent exists among different researchers on maximum magnitudes and other parameters for each fault of the NMSZ. The ESP seismic source model adopted results from different researchers by assigning magnitudes and corresponding weights to each fault. The maximum magnitudes and the corresponding weights are Mw 7.3 (0.4), 7.7 (0.5), and 8.1 (0.1) for the Blytheville Arch fault; Mw 7.4 (0.4), 7.6 (0.3), and 8.0 (0.1) for the Reelfoot fault; and Mw 7.0 (0.4), 7.4 (0.5), and 7.8 (0.1) for the East Prairie fault. The applicant modeled three faults rupturing in a cluster of events within a short period of time because the composite nature of sand blows uncovered in the field investigations suggests that the NMSZ earthquakes occurred as an event sequence. Based on combined paleoseismic and paleoliquefaction investigations conducted in the area, the applicant assigned recurrence time and probability for these event clusters of 200 years (0.1), 500 years (0.6), and 800 years (0.3).

One group of the contributors to the above magnitude estimates, Bakun, W.H. and M.G. Hopper, "Magnitudes and Locations of the 1811-1812 New Madrid Missouri, and the 1886 Charleston, South Carolina Earthquake," 2004, revised their magnitude estimates shortly after they submitted their first paper. In RAI 2.5.2-5, the staff asked the applicant to explain and quantify the impact on hazard (annual probability of exceedance (APE) of 10^{-5}) caused by the

revised magnitude estimates in Bakun and Hopper's latest publication. In its response, the applicant compared the magnitude estimates before and after the magnitude revision for the three faults of the NMSZ. Before the revision, Bakun and Hopper assigned the maximum magnitudes Mw 7.2, 7.1, and 7.4, respectively, to the New Madrid South, New Madrid North, and Reelfoot Rift. After the revision, the authors had two alternative magnitude models. In Model 1, the magnitudes for the New Madrid South, New Madrid North, and Reelfoot Rift are Mw 7.1 (6.8–7.9), 7.2 (6.8–7.8), and 7.4 (7.0–8.1), and in Model 2, the magnitudes for the New Madrid South, New Madrid North, and Reelfoot Rift are 7.6 (7.2–7.9), 7.5 (7.1–7.8), and 7.8 (7.4–8.1). The authors preferred Model 2. In response to this RAI, the applicant cited the research results from the work of the Exelon Generation Company (EGC) on the Clinton ESP site. After EGC interviewed Drs. Susan Hough, Bill Bakun, and Arch Johnston to obtain the latest information on the magnitude estimate, EGC revised the maximum magnitude assessment for faults within the NMSZ. This revised maximum magnitude estimate for the New Madrid South fault, the closest fault to the Grand Gulf ESP site, is Mw 7.53, compared to the weighted maximum magnitude of Mw 7.58 from this SSAR for the Grand Gulf ESP. The applicant concluded that the magnitude distribution and weighting provided in the SSAR captured the range of uncertainty recognized by the professional community. The EGC also performed a sensitivity analysis using various magnitude estimates on seismic hazard for the Clinton ESP site in Illinois, and the results of median and mean rock hazard for 1 hertz (Hz) hazard curves showed only a 3–4 percent increase because of the revised maximum magnitude estimates. Because the Clinton ESP site is located closer to the NMSZ than to the Grand Gulf ESP site, the applicant concluded that the impact of the magnitude revision to the Grand Gulf ESP site is insignificant. In addition, the applicant noted that this ESP application conservatively used only attenuation relationships for the midcontinent to estimate ground motion, although the ESP site is located inside the extended Mississippi embayment. Ground motions generally attenuate more rapidly within embayment than in midcontinent.

SSAR Section 2.5.2.1.2.4 discusses applying a moment-rate constraint on the large earthquakes (Mw greater than 7) of the NMSZ based on geodetic data. In RAI 2.5.2-2, the staff asked the applicant to justify the application of a moment-rate constraint on the modeling of those large earthquakes, in view of large uncertainties of the physical mechanism behind these earthquakes. The staff also asked the applicant to describe the impact for allowing ruptures into the lower crust, where shear wave velocities, and hence shear modulus, are higher than the shallow crustal values of California. Finally, the staff asked the applicant to explain and justify the assumption and reasoning leading to an estimated rupture area of 1300 square kilometers (km^2) for a rupture of the Reelfoot fault. In its response, the applicant stated that the moment-rate analysis in the SSAR was intended to provide the information regarding the relationship between maximum earthquake magnitudes and earthquake recurrences in the NMSZ and that the information was not used directly to assign or calculate weights for maximum earthquake magnitudes or recurrence intervals in the earthquake hazard analysis. The applicant recognized the uncertainty associated with estimates of the crustal rigidity (shear modulus) and down-dip geometry of the faults inside the NMSZ and that its analysis only intends to provide one model relating to earthquake magnitude and recurrence. However, the applicant stated that the parameters used in the analysis were derived through review of current published and unpublished literature, not from the moment-rate analysis. As a result of this RAI, the applicant deleted Section 2.5.2.1.2.4 from the SSAR.

<u>Saline River Source Zone Characterization</u>. SSAR Section 2.5.2.1.3 details the source parameters for the SRSZ. The applicant described the SRSZ parameters in the order in which

they appear in the logic tree of the model (see SSAR Figure 2.5-46)—(1) probability of existence, (2) source geometry, (3) characteristic earthquake magnitude, (4) earthquake recurrence, and (5) characteristic earthquake recurrence model. The applicant stated that the supporting evidence for the SRSZ includes coincidence of liquefaction, sparse seismicity, late Tertiary and possibly Pleistocene fault rupture, and geomorphic asymmetry of drainage basins. However, all these lines of evidence are not conclusive because nontectonic processes could cause an asymmetrical river basin, the background or distant earthquakes could also trigger liquefaction, and active faults are limited in number and in size. Based on this, the applicant assigned a 50-percent probability to the existence of the SRSZ. The applicant defined the SRSZ as an area source encompassing all the geomorphic, liquefaction, seismicity, and geologic evidence that suggests the existence of a localized seismic zone. The northwestern boundary of the SRSZ is defined based on the northern boundary fault of the Reelfoot Rift. The southeastern boundary is defined as the southward projection of the recently identified Reelfoot Rift-related marginal faults. The southwestern boundary is the southern-rifted margin of the North American craton, and the northern occurrence of basin asymmetry along the Arkansas River defines the northeastern boundary. The estimated magnitudes and corresponding weights for the source zone are 6.0 (0.3), 6.5 (0.6), and 7.0 (0.1) based on the extent of the liquefaction features and the observed surface faults. The applicant applied both characteristic and exponential earthquake recurrence models to the SRSZ, but assigned lower weight to exponential (0.1) and higher weight to characteristic recurrence (0.9). For the characteristic recurrence model, the applicant assigned a weight of 0.6 to the recurrence period drawn from paleoliquefaction data and a weight of 0.4 to the recurrence period drawn from fault slip rates. Finally, the applicant stated that the recurrence periods and weights assigned for the characteristic earthquake model varied from 399 (0.2) to 1725 (0.4) to 3500 (0.4) years for each magnitude variable based on paleoliquefaction data. Based on fault slip rate, however, the recurrence periods and weights were 10,000 (0.1), 2,000 (0.3), and 1,000 (0.6) years for magnitude Mw 6.0; 30,000 (0.1), 6,000 (0.3) and 3,000 (0.6) years for magnitude Mw 6.5; and 125,000 (0.1), 25,000 (0.3), and 125,000 (0.6) years for magnitude Mw 7.0.

In RAI 2.5.2-7, the staff asked the applicant to provide the reasoning behind assigning a higher weight (0.6) to the slip rate of 0.05 mm/yr, instead of 0.1 mm/yr for the SRSZ, and choosing the incision rate of 0.05 mm/yr over the other incision rates for the slip-rate estimation. In its response, the applicant stated that the SSAR wrongly reported the rate and weight pair; the correct weights are 0.1, 0.3, and 0.6 for the slip rates of 0.01, 0.05, and 0.1 mm/yr, respectively. The applicant will correct this error in the revised SSAR. The minimum slip rate of 0.01 mm/yr for the earthquake occurrence model is an approximation of 0.008 to 0.03 mm/yr obtained from paleoseismic studies and was assigned a low weight of 0.1. The applicant estimated a middle value of 0.05 mm/yr based on the combination of 0.03 mm/yr, derived from paleoseismic data, and a lower bound (LB) rate of 0.05 mm/yr derived from an incision rate of the 800–1300 ka intermediate Complex terrace. The applicant estimated the upper slip rate of 0.1 mm/yr based on the upper bound (UB) rate of 0.09 mm/yr from incision of the 800–1300 ka intermediate

Complex terrace and the LB rate of 0.3 mm/yr from incision of the 70–120 ka Prairie Complex. The high incision rates in the recurrence model are excluded because these rates, 0.8–1.7 mm/yr and 0.5 mm/yr, are not consistent with other lines of evidence used to approximate the fault slip rates. A number of factors may cause these high rates, such as hydrologic regime changes and mapping uncertainty.

<u>Effect of Updating the Earthquake Catalog on the EPRI-SOG Seismicity Parameters</u>. In SSAR Section 2.5.2.1.4, the applicant noted that, after it updated the EPRI-SOG catalog to include

events from 1985–2002, it found that the updated catalog (1) does not have any event greater than m_b 3.0 which occurred within about 110 miles of the ESP site, (2) does not show any earthquakes within the site region that can be correlated with known geologic structures, (3) does not show a pattern of seismicity that would require significant revision of the EPRI seismic source model, and (4) does not show any increase in the estimated rate of earthquake occurrences. After comparing the frequency of the earthquake occurrence of the original EPRI catalog with the updated catalog, the applicant concluded that it is not necessary to modify the original EPRI seismic parameters because the new catalog shows a slightly lower occurrence frequency. However, the applicant explained that it did not evaluate the seismicity for the NMSZ because it had already established a characteristic earthquake model for the NMSZ. In RAI 2.5.2-4, the staff asked the applicant to elaborate on the magnitude-frequency curve in SSAR Figure 2.5-47 regarding the exceeding magnitude values and the b value (a value used to describe the magnitude distribution) used in the curve. In its response, the applicant stated that the point on the magnitude-frequency curve corresponds to the frequency of exceedance of the magnitude value used to enter the curve, and the b value used to generate the magnitude-frequency curve is 0.91.

2.5.2.1.2 Site Probabilistic Seismic Hazard Analysis

Section 2.5.2.2 of the SSAR describes the procedure of PSHA for the ESP site in (1) source characterization, (2) magnitude conversion, (3) ground motion attenuation models, (4) LB magnitude, and (5) PSHA calculation.

<u>Seismic Source Characterization</u>. SSAR Section 2.5.2.2.1 describes the three seismic sources used in the PSHA for the ESP site. The applicant stated that the PSHA was performed using the EPRI seismic sources listed in SSAR Tables 2.5-8a through 2.5-8f and the new SRSZ and the updated NMSZ. The applicant further stated that it potentially double-counted seismic hazards by overlapping the new SRSZ and the characteristic NMSZ sources over the EPRI original seismic source model to provide a conservative estimate of the seismic hazards for the ESP site.

<u>Magnitude Conversion</u>. In SSAR Section 2.5.2.2.2, the applicant noted that it converted the m_b into Mw to apply the new EPRI attenuation relationships. The applicant used three m_b to Mw conversion relationships—Atkinson and Boore ("Ground Motion Relations for Eastern North America," 1995), EPRI (TR-102293, "Guidelines for Determining Design-Basis Ground Motions," 1993), and Johnston (A.C. Johnston, "Seismic Moment Assessment of Earthquakes in Stable Continental Regions-III. New Madrid 1811-1812, Charleston 1886 and Lisbon 1755," 1996)—with equal weights. In RAI 2.5.2-3, the staff asked the applicant to detail the steps and equations employed in the m_b to Mw conversion and identify where the conversion is performed in the hazard calculation. In its response, the applicant stated that it took the following four steps in converting m_b to Mw in its PSHA computation:

(1) Seismicity parameters for each seismic source are defined in either m_b or Mw. The input parameters for each m_b—Mw relationship also include the probability weight.

(2) As part of the magnitude integration, the applicant determined the occurrence frequency of earthquakes in a magnitude interval using original m_b or Mw input for a seismic source.

(3) The applicant will convert each m_b input into Mw before computing ground motions. The conversion will be made for the central m_b value in a magnitude interval.

(4) The applicant estimated ground motions for a given Mw and distance pair.

These procedures are repeated for each m_b input.

Ground Motion Attenuation Models. SSAR Section 2.5.2.2.3 describes ground motion models used in estimating rock motions at the ESP site. The applicant used the EPRI 2003 ground motion attenuation models. The EPRI 2003 ground motion models provide not only the estimate of the rock motions but also the alternative estimates of the median and aleatory uncertainty in ground motion for the midcontinent and the Gulf region of the CEUS. The applicant stated that the alternative models of the median and aleatory uncertainty, as well as their probability weights, represent the epistemic uncertainty in ground motions. The EPRI 2003 ground motion model is defined for different seismic source types, including general area sources, fault sources, and sources with large earthquakes (Mw greater than 7.0). Furthermore, fault sources can also be divided into rift or nonrift regions. Using the NMSZ as an example, the applicant stated that the midcontinent, rifted ground motion attenuation relationships should be applicable to the NMSZ. There are 12 estimates of the median ground motion, 4 estimates of aleatory uncertainty, and a total of 48 ground motion model estimates. For a general area source model, a total of 36 ground motion estimates exist.

Lower Bound Magnitude. SSAR Section 2.5.2.2.4 describes the LB magnitude used in the PSHA calculation for the ESP site. The applicant stated that it used Mw 5.0 as the LB magnitude to calculate earthquake hazards at the ESP site. This LB magnitude is consistent with the original EPRI 1986 source model.

Probabilistic Seismic Hazard Analysis Calculations. SSAR Section 2.5.2.2.5 describes the PSHA procedures and PSHA results for the ESP site. The applicant stated that it used the upgraded EPRI EQHAZARD software to calculate the seismic hazards for the ESP site. In SSAR Table 2.5-15, the applicant calculated seismic hazard results at different frequencies (0.5 to 25 Hz and peak ground acceleration (pga)) and presented the median 10^{-5} uniform hazard response spectrum (UHRS) for rock conditions at the ESP site. The applicant deaggregated the median hazards for low frequency (1–2.5 Hz) and high frequency (5–10 Hz), following the procedures of the RG 1.165, and presented the results in Figures 2.5.2-1 and 2.5.2-2. The applicant concluded that the results of deaggregation show that the majority of the contribution for both low- and high-frequency hazards is from the characteristic events associated with the NMSZ. The applicant also stated that to reduce the computing time it simplified the procedures of the PSHA by decreasing the number of branches inside the logic tree. The applicant evaluated the sensitivity of the seismic hazards resulting from the simplification and determined that the sensitivity is insignificant, which is demonstrated by the ESP median seismic hazard for spectral accelerations (Sa) of both 1 and 10 Hz in SSAR Figures 2.5-55 and 2.5-56.

In RAI 2.5.2-8, the staff asked the applicant to explain why the mean hazard curves for the frequencies 0.5, 1, 2.5, 5, 10, and 25 Hz are shown above or approximately coinciding with the 0.85 fractile curves for higher Sa (SSAR Figures 2.5-48 through Figure 2.5-54). In its responses, the applicant stated that the results of a PSHA in which the epistemic uncertainty is modeled produce a probability distribution on the estimate of the annual frequency of exceedance of ground motions. The applicant further stated that experience indicates that this

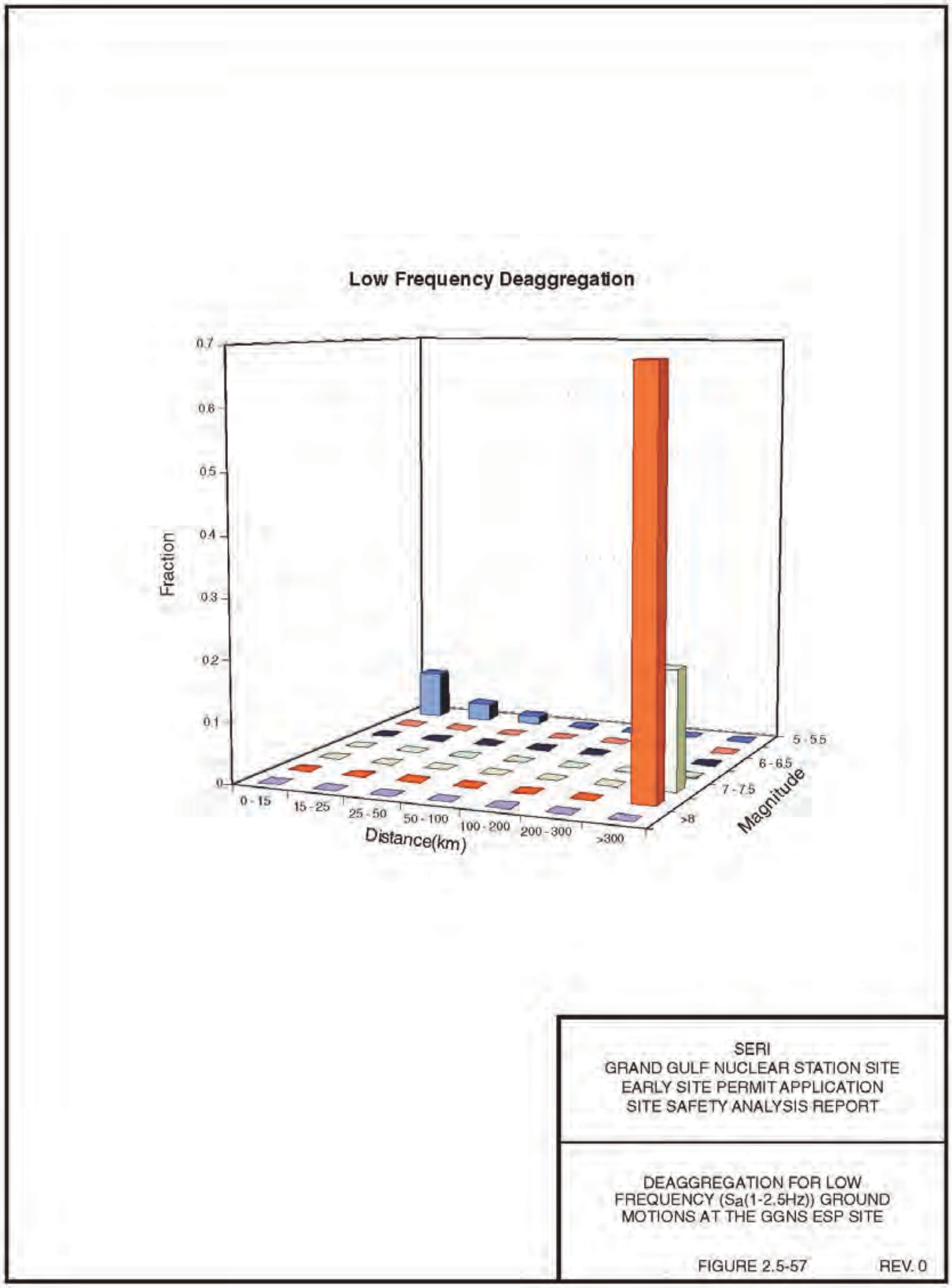
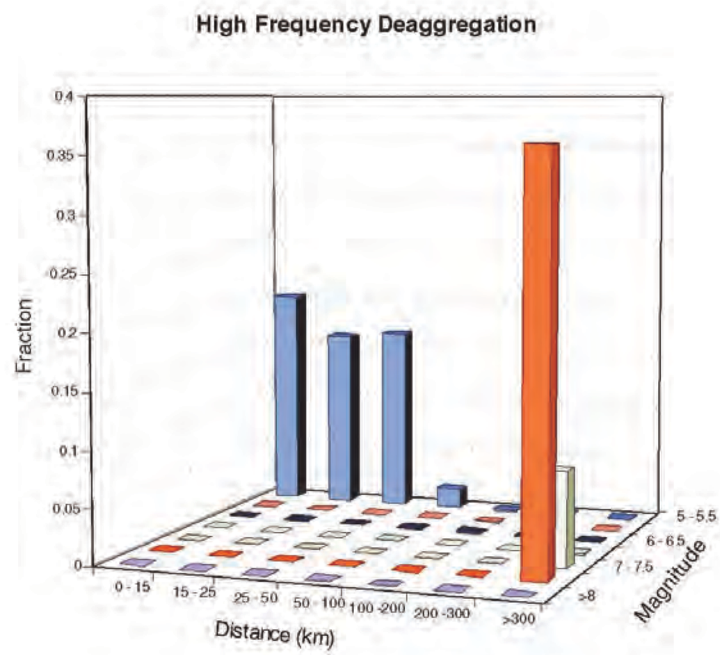


Figure 2.5.2-1 (SSAR 2.5-57) Deaggregation for low-frequency (Sa 1–2.5 Hz) ground motions at the GGNS ESP site



SERI
GRAND GULF NUCLEAR STATION SITE
EARLY SITE PERMIT APPLICATION
SITE SAFETY ANALYSIS REPORT

DEAGGREGATION FOR HIGH FREQUENCY (Sa(5-10Hz)) GROUND MOTIONS AND AT THE GGNS ESP SITE

FIGURE 2.5-58 REV. 0

Figure 2.5.2-2 (SSAR 2.5-58) Deaggregation for high-frequency (Sa 5–10 Hz) ground motions at the GGNS ESP site

distribution is not symmetric (i.e., not Gaussian) and is typically skewed and approximately lognormal in shape. As a consequence, the mean of this distribution is skewed (higher) with respect to the median and the degree of skewness is a function of epistemic uncertainty. As the epistemic uncertainty increases, the tendency is for the mean to correspond to higher fractile estimates of the frequency of exceedance.

In RAI 2.5.2-9, the staff asked the applicant to explain why the SRSZ, with a magnitude as large as 6.0–7.0 and a distance at 81–206 miles, makes no contribution to the deaggregation results. In its response, the applicant stated the following:

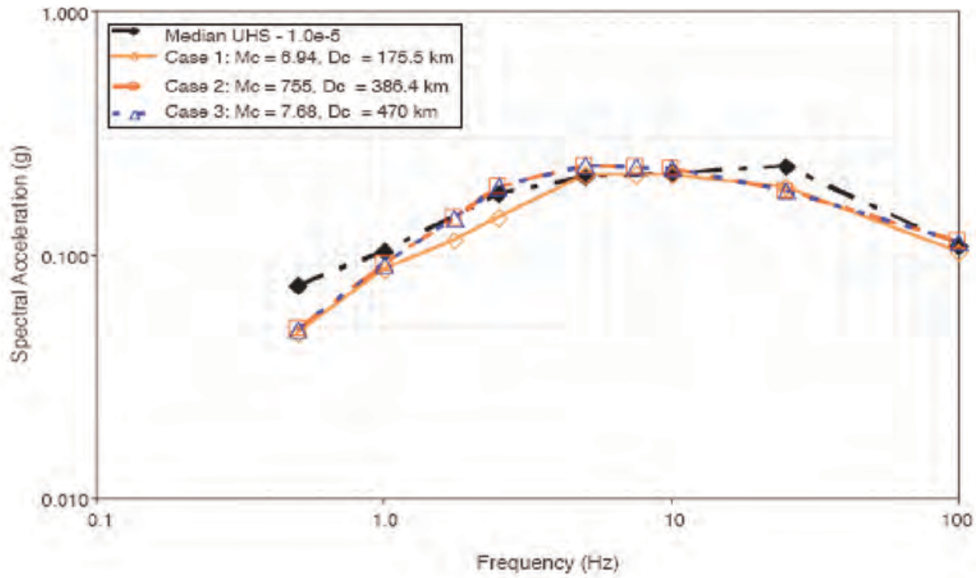
> The purpose of the magnitude-distance deaggregation is to provide a measure of the relative contribution of events (of different magnitude-distance pairs) to the total hazard. The deaggregation graphs in the SSAR show the relative contribution of earthquakes to the 10^{-5} median. The fact that events associated with the Saline River source (and all sources in this distance range) do not contribute to the 10^{-5} median hazard for the Grand Gulf site is primarily due to the fact that the overall rate of earthquake occurrences (Mw > 5) at these distances is extremely low. Further, the rate of occurrence of earthquakes of magnitude 6.0–7.0 is thus much lower. The result is that the estimated ground motion hazard at the Grand Gulf site is extremely low and, in a relative sense, makes little or no contribution to the 10^{-5} median hazard.

Controlling Earthquakes. SSAR Section 2.5.2.2.5.2 describes the controlling earthquakes for the ESP site. The applicant stated that it calculated the controlling earthquakes for low frequency (M=7.55 at 396.4 kilometers) and high frequency (M=6.94 at 175.5 kilometers) following the procedures of RG 1.165. The applicant also calculated the controlling earthquake for low frequency (1–2.5 Hz), but with distances greater than 100 kilometers (Mw 7.68 at 470 kilometers), because the contribution of large, distant events (greater than 100 kilometers) to low-frequency ground motions was greater than 5 percent. Using the EPRI 2003 ground motion model, the applicant determined the shape of the median response spectrum for each controlling earthquake. Figure 2.5.2-3 compares the controlling earthquake spectra and the UHRS. The applicant concluded that the response spectra from the low- and high-frequency-controlling earthquakes are a good approximation of the UHRS for frequencies greater than 1 Hz in terms of both shape and amplitude.

In RAI 2.5.2-10, the staff asked the applicant explain why the magnitude-distance bin, including the controlling earthquake, does not contribute to the high-frequency deaggregation. In its response, the applicant stated that the controlling earthquake for a given frequency range is the mean magnitude and mean distance of the deaggregation distribution. If deaggregation is multimodal or bimodal as in the Grand Gulf case, the resulting mean may not correspond to the magnitude values that are actually defined in the distribution.

2.5.2.1.3 Seismic Wave Transmission Characteristic of the Site

In SSAR Section 2.5.2.3, the applicant described the process to accommodate the surface soil effect on the incoming seismic waves at the ESP site. The applicant also briefly described the site-specific shallow soil profile and a generic deep shear wave profile extending to hard rocks referenced by the EPRI 2003 ground motion attenuation relationships.



SERI
GRAND GULF NUCLEAR STATION SITE
EARLY SITE PERMIT APPLICATION
SITE SAFETY ANALYSIS REPORT

COMPARISON OF THE GGNS 10^{-5} MEDIAN UNIFORM HAZARD RESPONSE SPECTRUM AND THE MEDIAN RESPONSE SPECTRA ASSOCIATED WITH THE CONTROLLING EARTHQUAKES

FIGURE 2.5-59 REV. 1

Figure 2.5.2-3 (SSAR Figure 2.5-59) Medium response spectra (10^{-5}) UHRS and controlling earthquakes

As stated in the section, the applicant calculated the ESP site ground motion using the EPRI 2003 ground motion attenuation relationships for CEUS hard rocks. The reference rocks for the EPRI 2003 attenuation relationships are defined as hard rocks with a shear wave velocity of 2.83 kilometers per second (km/s). Rocks with this kind of shear wave velocity only exist at depths exceeding 10,000 feet underneath the ESP site. However, as the applicant stated, the UHRS, and consequently the SSE, define 0.5 Hz as the lowest frequency (EPRI 1993 and PRI

EPRI 2003 ground motion models only extend to 2 seconds) and maximum amplification at 0.5 Hz for a typical deep firm soil profile at a depth of 1000 feet because the soil column to this depth already accommodates the amplification of the interesting frequencies. To be conservative, the applicant also stated that it used a local profile as deep as 3300 feet to calculate the soil amplifications. This local profile consists of a generic profile from about 225 feet to 3600 feet developed from the ground-shaking studies in the Mississippi embayment by the Mid America Earthquake Center (Professor Glenn Rix, personal communication, 2002) and a shallow profile from the surface to a depth of 225 feet based on three suspension log surveys. This shallow profile consists of 75 feet of loess, 85 feet of young alluvium, and 40 feet of old alluvium, as well as 25 feet of Catahoula formation, from the top to the bottom. The generic profile is relatively smooth, with shear wave velocities ranging from 250 meters per second (m/s) to 1000 m/s (see Figure 2.5.2-4).

The applicant also determined nonlinear dynamic material properties G/Gmax and hysteretic damping curves based on the laboratory testing of undisturbed samples taken during the survey. The applicant concluded that the laboratory dynamic test results were similar to the EPRI G/Gmax and hysteretic damping curves for cohesionless soils, which led to the adoption of the EPRI curves.

2.5.2.1.4 Site Response Analysis

In SSAR Section 2.5.2.4, the applicant described the methods it used to calculate the ESP site response and, in particular, the process to accommodate the variability in the shear wave velocity profile. The applicant used Approach 2A (use of two controlling earthquakes, both high and low frequencies, to derive soil spectra) recommended in NUREG/CR-6728 ("Technical Basis for Revision of Regulatory Guidance on Design Ground Motions; Hazard- and Risk-Consistent Motions Spectra Guidelines," 2001) to analyze the site response. The applicant used a controlling earthquake spectra of 1–2 and 5–10 Hz scaled to UHRS as control motions. It applied transfer functions, soil surface to hard rock outcropping, to each controlling earthquake in order to maintain the hazard level of the rock outcrop UHRS while incorporating variability in site-specific dynamic material properties. In order to fully count the uncertainty in the base shear wave velocity profile, the applicant incorporated 60 randomized profiles for each control motion for both shear wave velocity and layer thickness, and the randomization procedure itself came from a variance analysis of over 500 measured profiles. To accommodate variability in modulus reduction and hysteretic damping curves, these curves are also randomized around the base values. The applicant concluded that the resulting mean transfer function for each of two control motions reflects the best estimate (BE) effect of the soil/soft rock column. An upper and lower transaction of two standard deviations is used to prevent physically impossible modulus reduction or damping models. The mean transfer function, for each of two control motions (1–2 Hz and 5–10 Hz), accommodates site-specific variability in dynamic properties and depth to basement material. SSAR Figure 2.5-64 shows two transfer functions, with their envelope for the top of the loess, which correspond to 1–2 Hz and 5–10 Hz control motions. SSAR Figure 2.5-65, which is used to accommodate the effect of

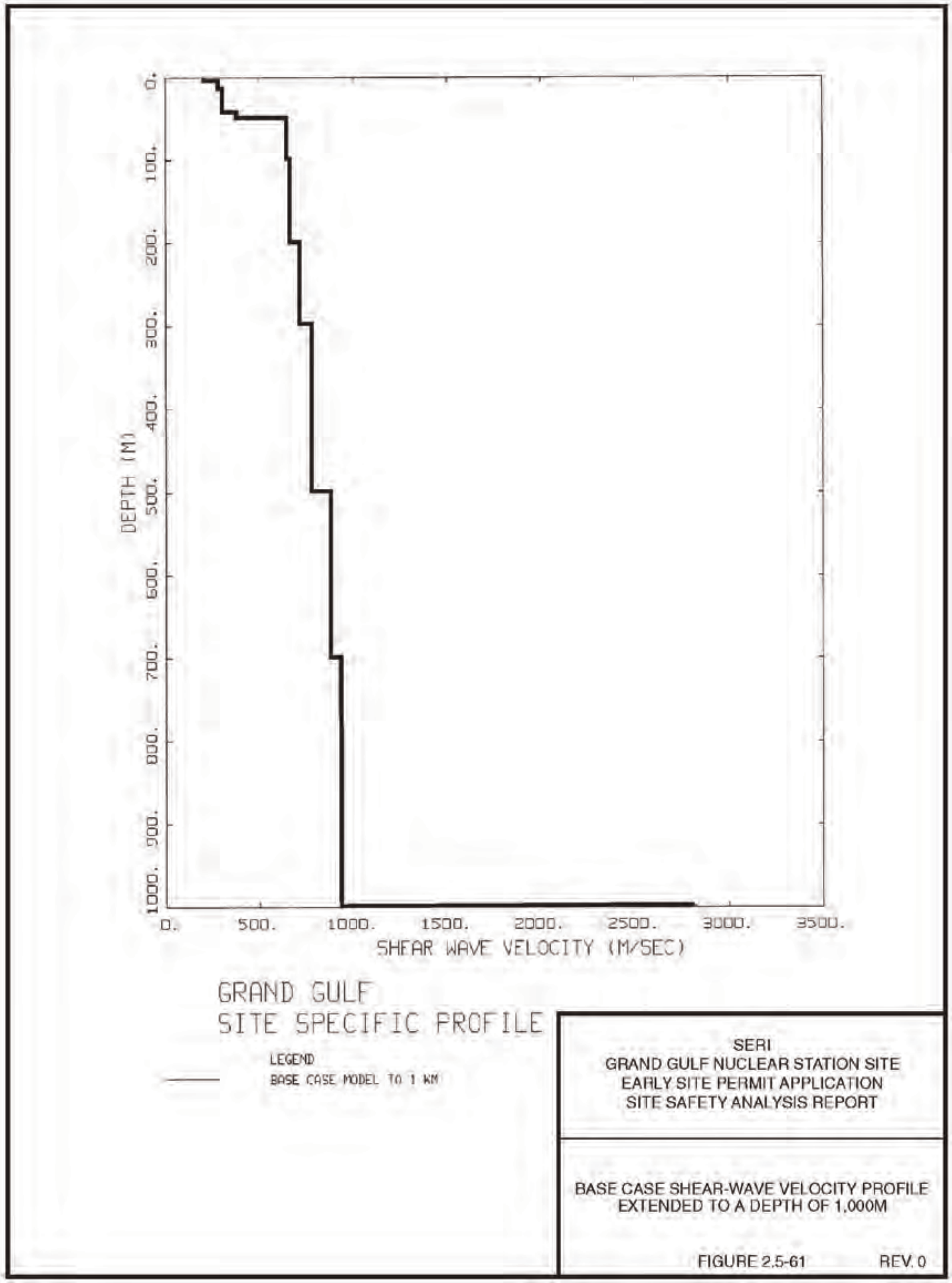


Figure 2.5.2-4 (SSAR Figure 2.5-61) Base shear wave velocity profile extended to a depth of 3300 feet

removing the surface loess (to the depth of 50 feet), shows the transfer functions for the top of surface with a shear wave velocity of 304 m/s.

In RAI 2.5.2-11, the staff asked the applicant to explain the basis for using Approach 2A instead of 2B (use of three earthquakes, including another central magnitude earthquake obtained from deaggregration to derive soil spectra) to derive soil spectra to generate the soil UHRS. The applicant responded that it considers the 2A approach to be appropriate because of the narrow magnitude range in earthquakes contributing to the UHRS at low and high frequencies. The magnitude difference is only 0.7 units, which makes the 2B approach unwarranted.

2.5.2.1.5 Safe-Shutdown Earthquake and Operating-Basis Earthquake

In SSAR Sections 2.5.2.5 and 2.5.2.6, the applicant described the procedures and relevant parameters to finally calculate the SSE ground motion and OBE.

The applicant stated that it enveloped the mean transfer functions for both the top of loess and the top of 1000 ft/s material and applied the envelope to the rock UHRS results in horizontal soil motions that are consistent with the median 10^{-5} APE hard rock UHRS. The applicant presented the calculated soil motions, together with the hard rock UHRS, in SSAR Figure 2.5-66. The applicant stated that the design horizontal motions are taken as the envelope of the two expected soil motions, as shown in Figure 2.5.2-5 below.

The applicant also stated that it used the vertical-to-horizontal (V/H) ratio in RG 1.60, Revision 1, "Design Response Spectra for Seismic Design of Nuclear Power Plants," December 1973, to estimate the vertical ground motions. The V/H ratio that is 2/3 at low frequency (0.1–0.3 Hz), increasing with the frequency to 1 near 8 Hz, is considered conservative. The vertical motions are shown in SSAR Figure 2.5-68.

The staff asked the applicant in RAI 2.5.2-12 to explain the basis for using the V/H ratio generated from the RG 1.60 design spectra and to compare the result to more recent recommendations for V/H ratio available in the literature. The applicant explained that V/H ratios are dependent on site conditions (rock versus soil), earthquake magnitude, and epicentral distance. The empirical V/H ratios for deep soils increase with magnitude and decreasing epicentral distance within 31 miles. At about 31 miles, the empirical ratio of deep soils at magnitude 7.5 has a maximum of about 1 near 15 Hz, decreasing to about 0.5 at 2 Hz and below. At distances greater than 31 miles, the V/H ratios decrease somewhat at high frequency with increasing distance and remain relatively constant (at about 0.5 Hz) at low frequency. For source distances beyond 31 miles, the RG 1.60 V/H ratio is considered to be a conservative estimate of site-specific vertical motion.

The applicant stated that the OBE ground motion spectrum is assumed to be one-third of the SSE spectrum, according to Appendix S to 10 CFR Part 50.

2.5.2.2 Regulatory Evaluation

SSAR Section 2.5.2 presents the applicant's determination of ground motion at the ESP site that could result from possible earthquakes that might occur in the site region and beyond. According to the SSAR, the applicant's assessment addresses the requirements in 10 CFR

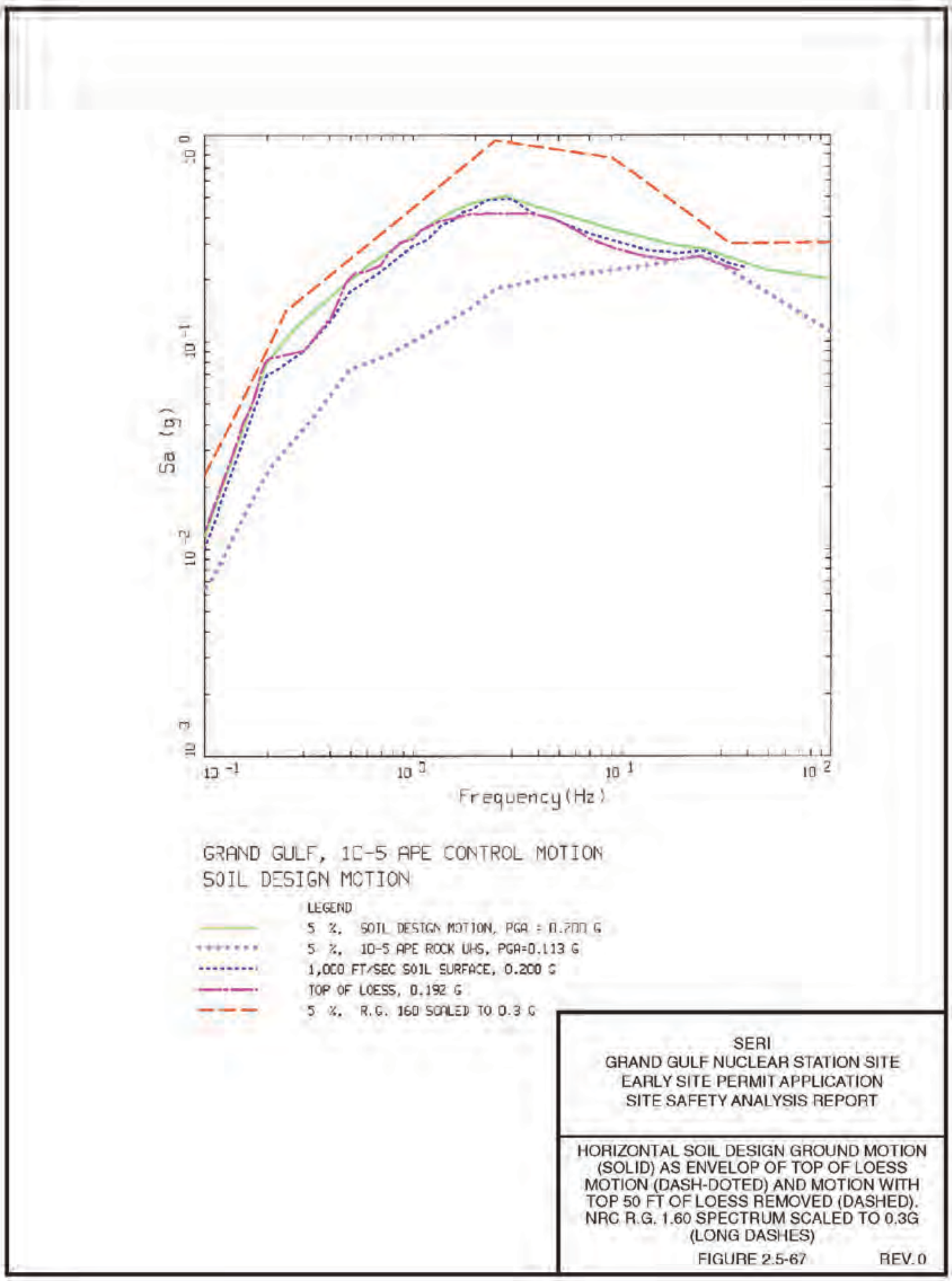


Figure 2.5.2-5 (SSAR Figure 2.5-67) Horizontal soil design ground motion and ground motions with loess and without loess. RG 1.60 spectrum also included (scaled to 0.3 g)

50.34, “Contents of Applications; Technical Information,” Appendix S to 10 CFR Part 50, and 10 CFR 100.23. The applicant further stated that it developed this information in accordance with the guidance presented in Section 2.5.2 of Revision 3 of NUREG-0800 and RGs 1.70 and 1.165. The staff notes that the application of Appendix S to 10 CFR Part 50 in an ESP review, as referenced in 10 CFR 100.23(d)(1), is limited to defining the minimum SSE for design.

In its review, the staff considered the regulatory requirements of 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23(c) and (d), which require that the applicant for an ESP describe the seismic and geologic characteristics of the proposed site. In particular, 10 CFR 100.23(c) requires that an ESP applicant investigate the geological, seismological, and engineering characteristics of the proposed site and its environs with sufficient scope and detail to support estimates of the SSE and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. In addition, 10 CFR 100.23(d) states that the SSE for a site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface. Section 2.5.2 of NUREG-0800 provides guidance concerning the evaluation of the proposed SSE, and RG 1.165 provides guidance regarding the use of PSHA to address the uncertainties inherent in estimating ground motion at the ESP site.

2.5.2.3 Technical Evaluation

This section of the SER provides the staff’s evaluation of the seismological, geological, and geotechnical investigations that the applicant conducted to determine the SSE for the ESP site. The technical information presented in SSAR Section 2.5.2 resulted from the applicant’s surface and subsurface geological, seismological, and geotechnical investigations performed in progressively greater detail as these investigations moved closer to the ESP site. The SSE is derived from a detailed evaluation of earthquake potential, taking into account regional and local geology, Quaternary tectonics, seismicity, and specific geotechnical characteristics of the site’s subsurface materials.

SSAR Section 2.5.2 characterizes the ground motions at the ESP site which could result from earthquakes that might occur in the site region and beyond to determine the site SSE. The SSE represents the design earthquake ground motion at the site and the vibratory ground motion for which SSCs of a certain nuclear power plant must be designed to remain functional. According to RG 1.165, an applicant may develop the vibratory design ground motion for a prospective nuclear power plant using either the EPRI or LLNL PSHA for the CEUS. Following the recommendation of RG 1.165, the applicant adopted the EPRI PSHA in the ESP application. However, RG 1.165 also recommends that applicants perform geological, seismological, and geophysical investigations and evaluate any relevant research to determine whether revisions to the EPRI or LLNL PSHA databases are necessary. As a result, the staff focused its review on geologic and seismic data published since the late 1980s that could indicate a need for changes to the EPRI or LLNL PSHAs relative to the ESP site.

2.5.2.3.1 Seismic Source Characterization

The staff focused its review of SSAR Section 2.5.2.1 on the applicant’s description of the source model updates since the 1986 EPRI studies, with an emphasis on the seismic parameters of the characteristic NMSZ model and the new SRSZ. The applicant summarized

seismic source parameters used in the EPRI 1986 source model and concluded that the source model adequately captures the source information and uncertainty associated with new data and knowledge developed since the mid-1980s. The applicant stated that other than the characteristic NMSZ model and new SRSZ model, no new information would suggest significant modification to the EPRI seismic source model. After analyzing the EPRI earthquake catalog, the applicant also concluded that the new earthquake catalog actually provides a lower earthquake frequency than the one based on the original EPRI 1986 earthquake catalog.

In RAI 2.5.2-6, the staff asked the applicant to provide its reasoning for not updating the EPRI EST seismic source characterizations, following the 1994 Johnston studies, to give more weight to larger magnitude earthquakes for the seismic source surrounding the site. In its response, the applicant stated that the final results of Johnston 1994 on the seismic background source of the ESP site do not significantly change the source parameters of the 1989 EPRI-SOG seismic source model. The staff reviewed the applicant’s response to the RAI and found that it does not adequately address the recent change and the impact of this change on the original EPRI seismic source model in terms of the regional seismic background source for the ESP site. In the original EPRI seismic source model, each team had outlined and assigned both magnitudes and associated weights to the Gulf Coast region surrounding the ESP site. The staff calculated the weighted maximum magnitude for the Gulf Coast region, using EPRI source model parameters provided by the SSAR text and tables, and obtained an effective maximum magnitude of Mw 5.0 for the Grand Gulf background source for all the ESTs, giving equal weight to each of the six ESTs. The Grand Gulf Basin, with its transitional basement structure and relatively undisturbed sedimentary rocks, is relatively stable. Nevertheless, its immediate neighboring structural unit, the Ouachita Orogenic Belt, is also considered as inactive in terms of tectonic activity because, “although many large Paleozoic thrust faults of regional extent are mapped through the Ouachita Orogenic Belt, none display geological evidence of Quaternary activity.” However, the latest paleoseismic research on the SRSZ has proved that even within this relatively nonactive tectonic unit, it is still possible to uncover a new seismic source. Based on the combined evidence of geomorphic, paleoliquefaction and Quaternary faulting of the SRSZ, as well as the closeness of the SRSZ to the ESP site, the applicant has incorporated the SRSZ into the ESP seismic hazard calculation, although the lines of evidence are not conclusive. The applicant assigned maximum magnitudes and weights of Mw 6.0 (0.3), 6.5 (0.6), and 7.0 (0.1) and a return period varying between 390 and 125,000 years to the SRSZ, depending on the source of determination and magnitude distribution. The weighted maximum magnitude for the Ouachita Orogenic Belt, using the parameters provided by the SSAR, is Mw 5.4, which is also much less than the weighted maximum magnitude of the SRSZ. The discovery of the SRSZ has proven that the original estimates of the EPRI 1986 project are relatively low for the Ouachita Orogenic Belt. Although the SRSZ is not located inside the Gulf Coast Basin, the Gulf Coast Basin also has a faulted passive margin beneath the site, and the ESP site is in the midst of the Gulf Coast plain. SSAR Figure 2.5-16b shows that the crust beneath the site region has been thinned by Mesozoic extension and the upper crust thins mainly by extensional faulting. The thinning is probably caused by the extensional passive-margin faulting. Global observations imply that at least some extensional passive-margins remain able to cause relatively large earthquakes. The staff considers that all these have demonstrated the need to update the background source parameters for the ESP site. The staff concluded that the applicant’s response did not adequately address this update to the original EPRI-SOG background seismic source for the ESP site.

In responding to above issue addressed in Open Item 2.5-1, the applicant stated in its submittal dated December 10, 2004 (Entergy, Response to Request for Additional Information Letter No. 5), that initial results of the Johnston 1994 EPRI studies were available to the ESTs in a report titled “Methods for Assessing Maximum Earthquakes in the Central and Eastern United States,” by Coppersmith et al. (1987). The observation that the largest stable continental region (SCR) earthquakes appear to be associated with the Mesozoic and younger crust was known to the ESTs. Several ESTs explicitly referred to the preliminary worldwide database in their estimate of maximum magnitude for seismic sources in the CEUS. The ESTs used a variety of approaches and philosophies to estimate maximum magnitude and incorporated large uncertainties in their estimates. The applicant also indicated that the tectonic stability is another reason for not assigning high weights to larger magnitude. Up to 10,000 feet of unfaulted Cretaceous and younger sediments document the absence of capable tectonic structures in the site vicinity. In addition, the applicant stated that the SRSZ is a unique seismic source for the following reasons:

- The SRSZ is located in an area underlain by continental crust and a Paleozoic fold and thrust belt.

- It overlies the southern projection of the Reelfoot Rift.

- It contains geomorphic lineaments suggestive of long-term tectonic deformation.

- It contains localized Tertiary faults and folds associated with a pattern of increased microseismicity.

- It contains mid-Holocene-age liquefaction features.

The applicant emphasized that these unique geologic, geomorphic, and seismologic features were not observed elsewhere in the Gulf Coast Basin and are not present in the vicinity of the ESP site. The applicant also indicated that all the background seismic sources in the EPRI model that contributed significantly to the hazard at the Grand Gulf ESP site were revised to have a minimum magnitude of 5.0 or greater.

After reviewing the applicant’s response to the open item, the staff concurs with the applicant that the background source for the GGPN has been updated because ESTs had the access to the preliminary version of the Johnston 1994 studies and considered their view on the potential maximum magnitude of the Mesozoic or younger extended crust zone. Since the ESTs were facilitated in a system similar to the one recommended by the Senior Seismic Hazard Advisory Committee (SSHAC, 1997) in NUREG/CR-6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts,” issued November 1997, different teams chose to characterize their own seismic sources with different maximum magnitudes based on expert opinions. The applicant did not assign higher weights to the larger magnitudes because of the absence of capable tectonic features in the site vicinity, including the lower seismic activity and nearly 10,000 feet of undisturbed Cretaceous and younger sediments. The staff also concurs with the applicant that the SRSZ is unique in its tectonic background and different from that of the ESP site. Based on the applicant’s response, the staff concludes that this open item is resolved.

In RAI 2.5.2-5, the staff also asked the applicant to explain and quantify the impact caused by the revised magnitude estimates in Bakun and Hopper's latest publication. In its response, the applicant cited results from a sensitivity analysis using updated magnitude estimates on seismic hazard at the Clinton ESP site in Illinois performed by EGC. The applicant stated that, based on the fact that the results of median and mean rock hazard for 1-Hz hazard curves increased only 3–4 percent as a result of the magnitude changes to the Clinton ESP site and because the Clinton ESP site is closer to the NMSZ than the Grand Gulf ESP site, the impact of the magnitude revision to the ESP site is insignificant. The staff concurs with the applicant in concluding that the impact caused by revising the maximum magnitude of the NMSZ earthquakes is insignificant because the sensitivity test for the Clinton ESP site showed only a 3–4 percent increase in seismic hazard level.

Based on its review of SSAR Section 2.5.2.1 and the applicant's responses to the RAIs and open items, the staff finds that the applicant has adequately characterized the overall seismic sources at the ESP site. The staff also concludes that the applicant's descriptions of the NMSZ and the SRSZ are accurate and sufficient in addressing the need for calculating the SSE for the ESP site. In addition, the staff concurs with the applicant's decision to use the original EPRI seismicity parameters based on its comparison of the updated seismic catalog with the original EPRI catalog.

2.5.2.3.2 ESP Probabilistic Seismic Hazard Analysis

To evaluate the applicant's PSHA procedures and results, the staff reviewed the information presented in SSAR Section 2.5.2.2. The staff focused its review on the ground motion model and magnitude conversion relationships used in the PSHA process and subsequent results.

In describing ground motion attenuation models, the applicant stated that it used attenuation models developed by an EPRI-sponsored project in 2003. This is an SSHAC Level 3 analysis, sponsored by EPRI, to update ground motion attenuation studies in the CEUS using the latest strong motion data. The EPRI 2003 attenuation models used various attenuation relationships and their clusters to estimate ground motions for rock sites, including not only the medians, but also the aleatory uncertainties. In addition, the EPRI 2003 ground motion models classified the attenuations based on earthquake sources, paths, and event sizes. NUREG/CR-6372 (SSHAC, 1997) provides the guidelines for performing this analysis. Because the 2003 attenuation model was used extensively in each ESP application, the staff studied the attenuation model and addressed its concerns in the Open Item 2.5-2.

The first issue in the Open Item 2.5-2 is related to the weighting scheme of the EPRI 2003 ground motion. The EPRI ground motion study used 13 different ground motion attenuation relationships, grouped into four clusters. For cluster 1, EPRI gave the highest weight (0.90) to the three attenuation relationships reported by Silva, W.J., N. Gregor, and R. Darragh, "Development of Regional Hard Rock Attenuation Relations for Central and Eastern North American, Pacific Engineering and Analysis," 2002. However, the applicant did not provide plots or tables of the residuals as a function of attenuation relation, magnitude, distance, and frequency. Similarly, for clusters 2 and 3, the ground motion experts applied higher weights to different attenuation relationships within each cluster. The staff concludes that the applicant needs to provide the rationale for these weights. In addition, the EPRI ground motion model assigned different weights to each of the four clusters. The weights were given after the expert

panel members, convened for the EPRI ground motion study, subjectively evaluated how well the alternative ground motion models relied on seismological principles. Because the evaluation criteria were not provided, the staff was unable to evaluate the weights applied to the four clusters.

In responding to the above issue, the applicant provided the staff with tables of statistics that compare each of the ground motion relationships and the CEUS earthquake database. The applicant also provided plots of residuals for each of the cluster 1 ground motion models and plots comparing the final overall cluster 1 model to the actual CEUS earthquake data. The applicant stated that the mean and variance were computed by assigning weights to the models within a cluster. These weights were determined on the basis of a model's consistency (mean deviation from the data) with available strong motion data. The staff examined the plots and tables of model residuals provided by the applicant for the cluster 1 ground motion models. The staff verified that, for the ground motion frequencies (1, 5, and 10 Hz), the three Silva et al. ground motion models do provide the smallest mean residual values (i.e., best fit to the earthquake strong motion data) compared to the other cluster 1 models. As a result, EPRI gave weights of 0.192, 0.148, and 0.560 to these three ground motion models. Since the EPRI 2003 expert panel members gave the three Silva et al. attenuation relationships the highest overall weight (0.9) in the cluster, a subsequent concern is that since all three Silva et al. attenuation relationships have the same wave propagation travel path terms and parameters, this limited path variability could have biased the overall cluster 1 ground motion model. To resolve the staff's concern, the applicant responded as follows:

> The ground motion models in Cluster 1 considered a range of alternative stress drop models and alternative Q and path models. Collectively, these models represent alternative single-corner [shape] source spectrum models for the CEUS. In aggregate, these models provide a measure of the epistemic [modeling] uncertainty in the median ground motion based on the single-corner source spectrum models (e.g., intra-cluster variability).

In responding to the staff's concern regarding the criteria to assign weights to four clusters, the applicant provided the following explanation in its submittal dated June 21, 2005 (Entergy, CNRO-2005-0000-DRAFT):

> The final step in the evaluation of the median ground motion models was the assessment of relative weights for the four clusters. Each cluster represents an alternative approach for modeling earthquake ground motions in the CEUS. By assigning a weight to each cluster a final distribution and estimate of the epistemic uncertainty on the median ground motion was determined. To develop the relative weights for each cluster the following were considered:
>
> - consistency of the cluster median (mean log ground motion) with CEUS strong motion data,
> - the strength of the seismological principles used by the models in a cluster, and
> - the degree to which modeling of epistemic uncertainty was considered in developing individual ground motion relationships.

The second issue in the Open Item 2.5-2 is related to the Q value. The Silva et al. cluster 1 relationships use an expression for the seismic attenuation parameter, Q, which is frequency dependent. This frequency-dependent Q value was derived from an inversion of the data from the 1988 Saguenay earthquake. This inversion solves for Q, as well as the local site attenuation parameter kappa and the stress drop, which is the difference between the initial stress before the earthquake and the final stress. The staff was unable to determine how the recordings from a single earthquake can provide well-resolved values of both crustal Q and site kappa. The Q value of 317 at 1 Hz is much lower than values found in other studies of eastern

North American earthquakes. In addition, other studies have found less frequency dependence of Q in the east than in the west, which is contrary to the findings of Silva et al.

In responding to this staff's concern, which is related to the Silva et al. cluster 1 attenuation relationships, the applicant stated the following:

> The model functional form, basis for parameter selection, and the results developed in Silva et al. (2002) and its predecessor, Silva et al., ["Characteristics of Vertical Strong Ground Motions for Applications to Engineering Design,] (1997), are the responsibility of the lead author. Of particular relevance is the interdependence between model parameters, how the parameters were determined, model sensitivity to its parameters, and reasonable ranges in parameter values, based on expert judgement and expert interpretation of the scientific literature. It is unclear if a summary justification for the results of the Silva et al. (1997 and 2002) studies would resolve the items identified that seem, ultimately, to represent differences in expert judgement.
>
> Differences in expert judgement are often difficult to reconcile. For this very reason, the SSHAC process was developed and accepted for use by the NRC. The EPRI 2003 ground motion model was developed by implementing a SSHAC Level 3 assessment process during which the EPRI Expert Panel identified the Silva et al. Relationships as ones that should be included in the assessment and evaluated. The EPRI Expert Panel considered specific parameterizations of individual ground motion relationships in determining whether or not a relationship should be included in the SSHAC Level 3 assessment process. All ground motion relationships identified as viable by the Expert Panel were evaluated using the same criteria following the SSHAC Level 3 process.
>
> The SSHAC process does not guarantee that every scientist will agree with the assessments. It is rather intended to assure that the assessed results reflect the preponderance of current scientific views, which is the underpinning of safety decisionmaking.

After reviewing the applicant's responses to the concerns raised in Open Item 2.5-2, the staff concludes that the applicant has adequately explained why it assigned different weights to various attenuation relationships inside a ground motion cluster, and to four ground motion clusters, and also explained that the selection of Silva et al. Cluster 1 attenuation relationships that uses a frequency dependent Q value was determined by the expert panel established based on SSHAC Level 3 process. The staff concludes that the applicant has adequately

resolved each of the staff's concerns with regard to applying the EPRI 2004 ground motion models to the ESP site. Therefore, this open item is resolved.

SSAR Section 2.5.2.2.2 and the applicant's response to RAI 2.5.2-3 describe in detail the procedure to implement the magnitude conversion from body-wave magnitude to moment magnitude within the hazard computing procedures. The staff finds the response sufficient in explaining the steps and procedures, as well as the magnitude conversion relationships (including parameters) used in the PSHA. The staff concludes that the applicant's procedures and relationships used in converting body-wave magnitude to moment magnitude are appropriate.

The applicant described the PSHA results in SSAR Section 2.5.2.2.2, including the rock UHRS and the deriving controlling earthquakes for both low and high frequencies. In response to RAI 2.5.2-8, the applicant explained the asymmetry of the mean hazard curves relative to the median hazard curves for high Sa. The staff concurs with the applicant's conclusions that the degree of skewness is a function of epistemic uncertainty and that the increased uncertainty at the ESP site is caused by a number of factors, including uncertainties associated with earthquake magnitude conversions and the new EPRI 2003 ground motion model.

In RAI 2.5.2-9, the staff asked the applicant to explain why the magnitude and distance bins corresponding to the SRSZ make no contribution to the deaggregation result, although the SRSZ maximum earthquake magnitude is as large as 6.0–7.0 and at a distance of 81–206 miles from the ESP site. The applicant stated that the overall rate of earthquake occurrence (Mw greater than 5) at these distances for the SRSZ was too low to contribute to the 10^{-5} median hazard. However, the staff notes that in SSAR Section 2.5.2.1.3 and in its response to RAI 2.5.1-3, the applicant stated that the activity of the SRSZ has a probability of 0.5 for its existence. This probability of existence implies that the SRSZ has a 50 percent chance to produce an earthquake with a magnitude between 6.0–7.0. The applicant also assigned three different return periods (388, 1725, and 3505 years) to the characteristic SRSZ earthquake model based on paleoliquefaction data. As such, a SRSZ earthquake with a 3,505-year return period should repeat about 30 times during the 100,000-year time span that is associated with a 10^{-5} probability of exceedance. Therefore, the staff was not certain why the magnitude and distance bins corresponding to the SRSZ do not contribute to the hazard at the 10^{-5} probability level.

In responding to the staff's concerns addressed above, the applicant further provided more details on its method for adding the SRSZ to the original EPRI-SOG seismic source model. The applicant stated that it did not modify the geometry and recurrence parameters of the original EPRI-SOG model determined by the six EST teams, to accommodate the SRSZ. Instead, the applicant retained the original EPRI-SOG model and simply added the SRSZ to the original source. The applicant stated that the addition of the SRSZ double-counts the seismicity in the geographic region defined by the SRSZ. However, the applicant emphasized that the SRSZ has only a 0.5 probability of activity (defined as probability of existence in the SSAR), because of the nonconclusive seismic evidence associated with the SRSZ. This 0.5 probability of activity for the SRSZ resulted in a logic tree source model with half of the branches consistent with the original EPRI-SOG seismic source model and the other half consistent with the original model plus the SRSZ. Since the SRSZ is about 130 to 329 kilometers from the ESP site, events that contribute to the site seismic hazard are those with magnitudes greater

than 6.0. Based on the updated EPRI-SOG seismic characterization model that includes the SRSZ, there is only a 0.35 probability of earthquakes larger than 6.0 (0.5, probability of the existence, times 0.7, probability for earthquakes larger than 6.0). The applicant stated that as a result of this low probability (0.35), more than 0.5 of the weight in the logic tree is associated with events that do not have a maximum magnitude greater than 6 at the distance associated with the SRSZ. The applicant also concluded that since the earthquake that would contribute to the hazard at these distances does not occur, the median hazard in the distance bins is zero.

After reviewing the applicant's response, the staff concluded that it does not adequately address the absence of the SRSZ's contribution to the deaggregated seismic hazard at the ESP site. The absence of the hazard contribution from the SRSZ is probably due to the applicant's computational procedure for this particular seismic zone, which has a 0.5 probability of activity. To confirm this conclusion, the staff performed an independent evaluation using seismic sources similar to those used by the EPRI-SOG for the ESP site. For its evaluation, the staff also used a source weighting scheme similar to that used by EPRI -SOG and added a new source similar to the SRSZ. The staff then calculated and deaggregated its seismic hazard using two methods. The first method, which is the method adopted by the applicant model, resulted in no contribution from the SRSZ to the deaggregated seismic hazard result. The second method resulted in the appearance of a contribution from the SRSZ to the site hazard. The second method differs from the first one in that, instead of using those magnitude and distance bins associated with a 0.5 probability of non-activity in the deaggregation, they were omitted from the calculation. Implementing the second approach, the combined contribution from all relevant magnitude (M 6-6.5, M 6.5-7 and M >7.0) and associated distance bins are about 10 percent relative seismic hazard contribution for 5 and 10 Hz and 5 percent for 1 and 2.5 Hz, respectively. The staff's evaluation suggests that the SRSZ missing hazard contribution is from the applicant's computational procedure because more than half of the hazard values in the corresponding magnitude and distance bins for the corresponding non-activity logical tree branches are zeros. The staff asked the applicant to reevaluate the seismic hazard contribution from the SRSZ based on the staff's findings since the result could potentially affect the determination of the controlling earthquakes, and thus the determination of the SSE.

In response to the staff's evaluation, the applicant performed a sensitivity test to evaluate the impact of the missing SRSZ contribution to the controlling earthquakes. When increasing the hazard contribution from the SRSZ, the applicant balanced the total hazard by decreasing the hazard contribution from other sources, including the NMSZ, which ensured the NRC staff that the absence of the SRSZ contribution only caused by the applicant's deaggregation process. The applicant stated that the results from the sensitivity test indicate there is a very small change to the 5 and 10 Hz controlling earthquakes (see Table 2.5.2 -1) and associated median rock response spectrum. The calculated differences in response spectrum values ranges from +0.06 to -0.23 percent of the comparable Grand Gulf response spectra. The applicant concluded that even if the staff's approach were used to deaggregate the controlling earthquake, which shows that the SRSZ may contribute up to 5 to 10 percent of the median hazard, the contribution would not result in a significant change to the median rock ground motion at the Grand Gulf site. Based on the results of the applicant's sensitivity test and also the fact that the SRSZ is relatively far from the ESP site (130 to 329 km), the staff concurs with the applicant on its conclusion with regard to the specific situation of the ESP site. Therefore, this open item is resolved.

Table 2.5.2-1 Effect to controlling earthquakes when adding hazard contributions from the SRSZ

Frequency (Hz) of Controlling Earthquake	Magnitude and Distance (Assuming 5% contribution	Magnitude and Distance (Assuming 10% contribution	Original Controlling Earthquake Magnitude and Distance
5- 10 Hz	6.93	6.92	6.94
	174.46	173.44	175.5

The applicant's response to RAI 2.5.2-10 adequately explained why the high-frequency deaggregation graph shows no contribution from the magnitude-distance bin that contains the controlling earthquake.

Based on its review of SSAR Section 2.5.2.2 and the applicant's responses to the RAIs as discussed above, the staff concludes that the applicant's description of the PSHA parameters and procedures for the ESP site is reasonably accurate and adequate. The staff concurs with the applicant on its conservative approaches in overlapping the new characteristic NMSZ onto the original EPRI source model, as described in this section, and in using only attenuation relationships for the midcontinent to estimate ground motion, although the ESP site is located in the extended Mississippi embayment.

2.5.2.3.3 Seismic Wave Transmission Characteristics of the Site

The staff's review focused on the applicant's description of the generic shear wave profile and parameters associated with the profile in SSAR Section 2.5.2.3. In RAIs 2.5.4-4 and 2.5.4-6, the staff asked the applicant to explain the appropriateness of using the generic shear wave profile at the ESP site, including the associated parameters. Section 2.5.4 of this SER discusses the applicant's response to RAI 2.5.4-4. Based on its review of the contents of SSAR Section 2.5.2.3.3, the staff concludes that, other than the issues related to RAI 2.5.4-4, the applicant used an acceptable approach to characterize the site shear wave properties to the appropriate depth required by the reference rock used in the EPRI ground motion attenuation relationships in order to obtain the site-specific seismic wave responses. Therefore, the staff finds that the applicant's description of the site-specific seismic wave transmission characteristics is adequate and acceptable.

2.5.2.3.4 Site Response Analysis

The staff's review of this section of the SSAR focused on the applicant's description of the methodology used in deriving the site responses. The staff asked the applicant in RAI 2.5.2-11 to explain the basis for using Approach 2A instead of 2B to generate soil response spectra. The staff considers the applicant's response to be appropriate and sufficient, and, because of the narrow range in the magnitudes of the controlling earthquakes, it is appropriate to use Approach 2A. Based on its review of this section, the staff concludes that the applicant's description of the site responses and the approach used in deriving the site response are reasonably accurate and adequate.

2.5.2.3.5 Safe-Shutdown Earthquake and Operating-Basis Earthquake

Following its review of this and all the preceding sections of the SSAR, as well as the response to RAI 2.5.2-12, the staff considers the SSE developed for the ESP site to be consistent with Appendix S to 10 CFR Part 50, which defines the SSE as the "vibratory ground motion for which certain structures, systems and components must be designed to remain functional." The staff concludes that the applicant's approach, other than for those issues noted above, to calculating the SSE for the ESP site is also consistent with the requirements of 10 CFR 100.23(c) and (d) and RG 1.165. Therefore, the applicant's description of the SSE and the subsequent OBE is accurate and adequate.

2.5.2.4 Conclusions

As set forth above, the staff reviewed the seismological information submitted by the applicant in SSAR Section 2.5.2. On the basis of its review of SSAR Section 2.5.2 and the applicant's responses to the RAIs and open items, the staff finds 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 finds 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 provided in RG 1.165. The staff concludes that the controlling earthquakes and associated ground motion derived from the applicant's PSHA are generally consistent with the seismogenic region surrounding the ESP site. In addition, the staff finds that the applicant's SSE was determined in accordance with RG 1.165 and Section 2.5.2 of NUREG-0800. The staff concludes that the proposed ESP site is acceptable from a geological and seismological standpoint and meets the requirements of 10 CFR 100.23.

2.5.3 Surface Faulting

SSAR Section 2.5.3 describes the potential for tectonic fault rupture at the ESP site. The applicant concluded that since no capable tectonic sources that can generate both vibratory ground motion and tectonic surface deformation exist within a 5-mile radius of the ESP site, the site has no potential for tectonic fault rupture. SSAR Section 2.5.3.1 describes the applicant's geological, seismological, and geophysical investigations. SSAR Section 2.5.3.2 describes previous site investigations for the surface faults. SSAR Section 2.5.3.3 describes the geologic evidence, or absence of evidence, for surface deformation. SSAR Section 2.5.3.4 describes the correlation of earthquake epicenters with capable tectonic sources in the vicinity of the ESP site. SSAR Section 2.5.3.5 provides the characterizations of capable tectonic sources. Finally, SSAR Sections 2.5.3.6 through 2.5.3.7 describe zones of Quaternary deformation requiring detailed fault investigation and the potential for tectonic or nontectonic deformation at the site.

2.5.3.1 Technical Information in the Application

2.5.3.1.1 Surface Faulting Investigations

<u>Geological, Seismological, and Geophysical Investigations</u>. According to SSAR Section 2.5.3.1, the applicant performed the following investigations to assess the potential for surface faulting at and within a 5-mile radius of the ESP site:

- compilation and review of existing data
- interpretation of aerial photography
- discussions with current researchers in the area
- review of seismicity
- field reconnaissance

The applicant stated that a wealth of information is available for the site regarding the surface faulting studies. The information comes from three primary sources:

(1) previous research for the existing GGNS

(2) published and unpublished geologic maps from USGS, the State of Mississippi, and the University of Memphis

(3) seismicity data compiled from published journal articles and evaluated as part of this study

The applicant performed aerial and field reconnaissance investigations within a 5-mile radius of the ESP site. In particular, the applicant prepared an updated map of surficial deposits and geomorphology for the site location. The applicant also stated that it used the new map in combination with other preexisting maps to verify the absence of subsurface faulting or other forms of tectonic and nontectonic deformation by showing the surface of buried stratigraphic layers.

Previous Site Investigations. As noted by the applicant in SSAR Section 1.0, the existing GGNS site is very close to the ESP site, which is only 1200 feet west and 1000 feet north of the center of the GGNS containment. The applicant stated that the UFSAR summarizes the previous site investigations performed for the ESP site. The previous investigations illustrate the buried stratigraphic layers across the site using extensive subsurface data. These buried stratigraphic layers at the site include the Oligocene Glendon Limestone, Miocene Catahoula formation, and Pliocene and Pleistocene Upland Complex. The applicant concluded that, although these surfaces are eroded, they are not deformed by faulting, folding, or tilting across the site area. In addition, these undeformed surfaces document the absence of salt diapirs, collapse structures, volcanic intrusions, and other nontectonic deformations.

The applicant reevaluated Quaternary faulting studies by two renown geologists, Fisk and Krinitzsky (U.S. Army Corps of Engineers (USACE) Technical Memorandum No. 3-311, "Geological Investigation of Faulting in the Lower Mississippi Valley," issued 1950). Fisk postulated that a densely populated rectilinear fracture exists in the Mississippi Alluvial Valley. Krinitzsky mapped several hundred inferred faults in the same area based on physiographic evidence. He investigated several sites with closely spaced borings to verify the presence of faults in Tertiary deposits. However, the applicant pointed out that these geologists' views on active faults were affected by the then prevailing belief that a worldwide grid of fault patterns caused by planetary-scale influences dominates the earth's crust. Furthermore, the applicant

stated that detailed Quaternary mapping and numerous site-specific engineering geologic investigations disproved the existence of the Quaternary faults and fault zones across the site area that were proposed by Fisk and Krinitzsky. Two examples demonstrate this disapproval for some of the Quaternary faults. First, Fisk suggested that two possible fault zones may intersect about 3 miles north of the site near the mouth of the Big Black River. A cross-section constructed using borings indicates that the difference in the elevations of contacts, evidence for the faults, is attributable to the regional dip of stratigraphic units rather than a fault offset along the Big Black River. Fisk also proposed a lineament coincident with the Bayou Pierre. However, the continuation of stratigraphy across the site documented by the borings drilled both north and south of the bayou demonstrates the absence of the fault represented by the lineament. All of the previous investigations have disproved the existence of Fisk's fault zones and show no fault within 5 miles of the site.

New information developed since the original GGNS site investigation further confirms that no active faults exist within the 5-mile radius defined by the site area (USACE, "Geomorphology and Quaternary Geological History of the Lower Mississippi Valley," issued 1994).

<u>Geologic Evidence or Absence of Evidence for Surface Deformation</u>. SSAR Section 2.5.3.3 describes the detailed information regarding the existence of the Quaternary faults and their distance from the ESP site. The applicant reemphasized that no evidence exists of Quaternary fault offset in the site area. The applicant summarized that the closest Holocene active faults to the site are the growth faults, which are about 90 miles from the site. The closest Quaternary faults are the faults identified in the SRSZ, which are also about 90 miles from the site. The closest nontectonic sources, the Bruinsburg salt dome and the Gallaway salt dome, are within approximately 8.5 miles of the site.

<u>Correlation of Earthquake with Capable Tectonic Sources</u>. SSAR Section 2.5.3.4 states that no reported earthquake epicenters have been associated with faults within a 5-mile radius of the ESP site (site area). The closest earthquake of Mw 3.0 or larger is located 90 miles west of the site.

<u>Characterization of Capable Tectonic Sources</u>. SSAR Section 2.5.3.5 states that no capable tectonic sources exist within 5 miles of the ESP site. Subsurface borings completed for the existing GGNS document the absence of faulting in the site area, which is underlain by approximately 500 feet of Oligocene and younger deposits. These deposits decrease in elevation from north to south, are laterally continuous, and have a constant gradient across the site area. These deposits form part of a limb of a broad syncline structure, which has its axis following the current position of the Mississippi Alluvial Valley. The limbs of the syncline dip less than 1 degree toward the axis. This syncline is not a seismogenic feature, but it is caused by a slow isostatic adjustment of the crust to the continuous sediment loading.

Quaternary faults exist in the site region but all appear to lie at least 90 miles away from the site. The SRSZ is one of only two locations inside the site region where nonconclusive Quaternary faulting evidence has been found. The other location is the Gulf Coast area, where the growth faults are found. Tertiary faults of the Pickens-Gilberttown and Southern Arkansas fault zones are approximately 100 miles northeast of the site. The applicant repeated that no faults are mapped closer than 90 miles to the site.

Erosion occurred along several surfaces with gentle slopes, including the Oligocene Glendon Formation (30 Ma) and the Catahoula formation. No morphology for these surfaces or the younger surface of the Upland Complex has any indication of tectonic deformation.

The applicant also stated that the investigations by USACE (“Geological Investigation of Faulting in the Lower Mississippi Valley,” 1950) also support the conclusions that the Quaternary deposits in the site area are not faulted and previous mapped faults are not present. The USACE developed its geologic cross-section based on a series of borings along the Mississippi River.

Potential for Tectonic or Nontectonic Deformation at the Site. SSAR Section 2.5.3.7 states that the ESP site has a negligible potential for tectonic deformation. The applicant stated that, since the original studies in the early 1970s, no new information has been reported to suggest the existence of any Quaternary surface faults or capable tectonic sources within the site area. In addition, the site shows no evidence of nontectonic deformation, such as glacially induced faulting, collapse structures, growth faults, salt migrations, or volcanic intrusions.

2.5.3.2 Regulatory Evaluation

SSAR Section 2.5.3 describes the applicant’s evaluation of the potential for surface deformation that could affect the site. The applicant presented the information in SSAR Section 2.5.3 in accordance with the requirements of GDC 2, Appendix S to 10 CFR Part 50, and 10 CFR 100.23. The applicant also developed the geological, seismological, and geophysical information used to evaluate the potential for surface deformation in accordance with the guidance presented in Section 2.5.3 of NUREG-0800, Revision 3, and RGs 1.70, 1.132, 1.165, and 4.7. The staff notes that the application of Appendix S in an ESP review, as referenced in 10 CFR 100.23(d), is limited to defining the minimum SSE for design.

In its review of the application, the staff considered the regulatory requirements in 10 CFR 100.23(d)(2), which state that an applicant for an ESP must determine the potential for surface tectonic and nontectonic deformations. Section 2.5.3 of NUREG-0800 and RG 1.165 provide specific guidance concerning the evaluation of information characterizing the potential for surface deformation, including the geological, seismological, and geophysical data that the applicant must provide to establish the potential for surface deformation.

2.5.3.3 Technical Evaluation

This section of the SER provides the staff’s evaluation of the seismological, geological, and geophysical investigations carried out by the applicant to address the potential for surface deformation that could affect the site. The technical information presented in SSAR Section 2.5.3 resulted from the applicant’s surface and subsurface investigations performed in progressively greater detail as they moved closer to the ESP site. Through its review, the staff determined whether the applicant complied with the applicable regulations and whether the applicant conducted its investigations with an appropriate level of thoroughness.

In order to thoroughly evaluate the surface faulting investigations performed by the applicant, the staff sought the assistance of USGS. The staff and its USGS advisors visited the ESP site and met with the applicant to assist in confirming the interpretations, assumptions, and

conclusions presented by the applicant concerning potential surface deformation. Specific areas of review include the geological, seismological, and geophysical investigations (SSAR Section 2.5.3.1), previous investigations (Section 2.5.3.2), geological evidence or absence of evidence of surface deformation (Section 2.5.3.3), correlation of an earthquake with capable tectonic sources (SSAR Section 2.5.3.4), characterization of capable tectonic sources (SSAR Section 2.5.3.5), zones of Quaternary deformation requiring detailed fault investigation (SSAR Section 2.5.3.6), and the potential for surface tectonic deformation at the site (SSAR Section 2.5.3.7).

2.5.3.3.1 Surface Faulting Investigations

The staff focused its review of SSAR Sections 2.5.3.1 through 2.5.3.7 on the adequacy of the applicant's investigations to ascertain the potential for surface deformation that could affect the site. The staff reviewed the applicant's summary of previous site investigations recorded in the UFSAR and the recent investigations. The staff concludes that the applicant adequately investigated the potential for surface deformation in the site area. The staff and its USGS consultants also visited the site area and did not observe any evidence for Quaternary tectonic activity near the site. Therefore, the staff concludes that the applicant has adequately investigated the potential for surface deformation as required by 10 CFR 100.23.

Based on its site visit and its review of SSAR Section 2.5.3, as set forth above, the staff concurs with the applicant's conclusion that no evidence of Quaternary folding or faulting can be associated with these local faults.

2.5.3.4 Conclusions

In its review of the geological and seismological aspects of the ESP site, the staff considered the pertinent information gathered by the applicant during the regional and site-specific geological, seismological, and geophysical investigations. As a result of this review, described above, the staff concludes that the applicant performed its investigations in accordance with 10 CFR 100.23 and RG 1.165 and provided an adequate basis to establish that no capable tectonic sources exist in the site vicinity that would cause surface deformation in the site area. The staff concludes that the site is suitable from the perspective of tectonic surface deformation and meets the requirements of 10 CFR 100.23. In addition, the staff finds that the applicant appropriately considered the most severe surface deformation historically reported for the site and surrounding area, with sufficient margin for uncertainties, and satisfies GDC 2 in that respect.

2.5.4 Stability of Subsurface Materials and Foundations

SSAR Section 2.5.4 describes the characteristics of the subsurface materials and foundations at the ESP site. Section 2.5.4.1 of the SSAR describes the geotechnical characteristics of the site and the investigative programs conducted to support this characterization. SSAR Section 2.5.4.2 describes the site ground water conditions. Sections 2.5.4.3 and 2.5.4.4 of the SSAR summarize the soil response to dynamic loading and the evaluation of liquefaction potential at the ESP site. SSAR Section 2.5.4.5 describes the static stability conditions at the site, including an evaluation of bearing capacity and settlement. Finally, SSAR Section 2.5.4.6 briefly describes the geotechnical design criteria. Throughout this section, the applicant

referred to Engineering Report ER-02, “Geologic, Geotechnical and Geophysical Field Exploration and Laboratory Testing,” for details of the site’s geotechnical characteristics.

2.5.4.1 Technical Information in the Application

2.5.4.1.1 Detailed Site Investigation Program

SSAR Section 2.5.4.1 describes the static and dynamic engineering properties of the subsurface materials (between the surface and a depth of 200 feet) at the ESP site. This section also presents the laboratory testing program used to obtain the engineering characteristics of the subsurface materials.

<u>Description of Subsurface Materials</u>. The applicant stated, in SSAR Sections 2.5.4.1.1 and 2.5.4.1.2, that it derived the properties of the subsurface materials and the range of their thicknesses from four new borings made at the ESP site, together with a number of borings previously obtained as part of the original field exploration program conducted for the existing GGNS. Among the four new borings, two (B-2 and B-2a) are located at essentially the same (plan) location. The applicant did not take samples of the soil overburden at similar depth ranges in these two adjacent borings. Therefore, the applicant effectively made only three new borings as part of the ESP program. In addition, the applicant took four cone penetrometer tests (CPTs) to supplement the ESP site investigation. SSAR Section 2.5.1.2 cites the GGNS UFSAR, which states that the GGNS licensee drilled a total of 275 borings when constructing the existing GGNS. Those borings reach a maximum depth of about 450 feet below grade. The applicant included information from the previous investigation that is relevant to this ESP evaluation process.

The proposed ESP site is relatively flat with an elevation of 135 feet. The site is located immediately adjacent to the existing nuclear power plant on the bluff just to the east of the Mississippi River, with an area about 0.12 km^2 (30 acres). The applicant pressed the CPTs to a depth of about 100 feet and drove the borings to a depth of 180–240 feet. The combination of these recent data with previously available boring and sample data provides information on site stratigraphy to a depth of about 240 feet. The applicant stated that the remaining part of the site profile needed to determine seismic site response characteristics comes from other generic information available for the broad region around the site.

The applicant divided the shallow part of the subsurface materials into five zones, described as follows, from the surface downward:

(1) Localized fill—At various locations across the site, the GGNS licensee placed fill to relatively shallow depths to stabilize existing swales while constructing the existing GGNS. These relatively localized fills are no more than 20 feet thick and are unimportant to seismic site response analysis and to the foundation of the SSCs.

(2) Loess—Loess with a thickness of 55–85 feet forms the surface layer across the site. It is generally composed of relatively uniform inorganic silts of low to moderate plasticity (or an ML material according to the Unified Soil Classification System (USCS), with some silty clay intervals. The loess shows layering defined by difference in clay contents, color, shell content, and consistency. The engineering properties for different

loess layers do not show significant variation. Regionally and locally, the loess shows some minor cementation and soil structures that allow it to stand vertically in cuts and river banks. The CPT soundings show that the loess exhibits layering with a thickness from 6 inches to 40 feet. Standard penetration test (SPT) sample blow counts indicate that the loess is medium-stiff to stiff and has undrained shear strengths from 750 to 1500 psf. Measured shear wave velocities for the loess vary from 590 fps to 1450 fps.

(3) Upland Alluvium—Immediately below the loess is a zone of alluvium termed the Upland Alluvium, which consists of an interbedded sand and silty sand material with a USCS classification varying from SP (poorly graded sands or gravels with little or no fines) to SM (silty sands with little or no plasticity). Discontinuous layering ranges between 6 inches and 3 feet. Sand grains are subrounded to subangular, fine to medium grained, and consist of quartz with lesser feldspar and mafic lithologies. The Upland Alluvium is typically well sorted with low fines content and low plasticity. Some of this material may also contain plastic fines (an SC classification according to the USCS) interspersed in this zone. The Upland Alluvium is typically a medium-dense to dense formation that the CPT soundings penetrated to some extent. The thickness of the Upland Alluvium varies across the site from as little as 20 feet to as much as 100 feet. Undrained shear strength (ranging from 4000 psf to 8000 psf) of the Upland Alluvium is somewhat higher than that of the loess, and its shear wave velocities vary from 740 fps to as much as 1750 fps.

(4) Old Alluvium—The Old Alluvium consists of interbedded clayey sands, sandy clay, silty sand, and gravelly sand. The Old Alluvium is poorly to well sorted and typically exhibits much poorer grading than the overlying Upland Alluvium. The Old Alluvium also exhibits layering with a thickness between 3 inches and 4 feet. Gravel-size clasts include a large percentage of soft clay and claystone rip-up clasts. Finer grained layers of the Old Alluvium exhibit low to moderately high plasticity, and the Old Alluvium contacts with the overlying Upland Alluvium unconformably. It is generally more difficult to drill into the Old Alluvium than either the loess or the Upland Alluvium using standard boring equipment. The SPT blow counts in available samples indicate the Old Alluvium to be dense to very dense. The few profiles presented in the SSAR indicate that the layer extends to a depth of approximately 200 feet. The applicant also noted that measured shear wave velocities of the Old Alluvium vary from as little as 530 fps to as much as 3360 fps. Cross-sections across the site show that the Old Alluvium appears as lenses between the overlying Upland Alluvium and underlaying Catahoula formation.

(5) Catahoula formation—The Catahoula formation consists of gravelly sands, hard clays, and claystone. The claystone is highly plastic, indicating some fracturing characteristics encountered in samples or recovered core, and possesses some slaking characteristics when soaked in water for several minutes. Based on SPT blow count correlations, the Catahoula formation (blow counts = 82) is defined as hard to very hard and is classified as a soft rock-like material. Its shear wave velocities vary from 1500 to 2830 fps. The applicant did not state the anticipated bottom depth of the Catahoula formation because it was only encountered in one of the deepest borings, which penetrated to a depth of 240 feet.

In RAI 2.5.4-2, the staff asked the applicant to describe the characteristics of the fill material and controls, if any, placed on the fill at the time of its placement. In its response, the applicant noted that before the construction of the GGNS, a system of steep-walled drainage swales crossed portions of the ESP site. The licensee filled these swales during site grading to form the present upper (Elevation 155 feet) and lower (Elevation 134 feet) pads that are encompassed by the ESP site and PPBA. The applicant discussed these localized fills in the SSAR but did not show the extent of the filled swales. Therefore, the applicant will modify SSAR Figure 2.5-69 to more accurately depict the extent of the swale fills. The modified figure will also show that the bottom of the swale is located about 30 to 50 feet below the present grades. The applicant also stated that the UFSAR for the existing GGNS site discusses the engineered structural fill placed at the powerblock of the existing plant but does not discuss the engineering controls used for fill placement at the ESP site. The SPT blow counts in the fill from an ESP boring located in the center of one of the fill areas range from 5 to 7. These blow counts are less than the underlying native loess (blow counts from 11 to 13) but are within the range of loess soils in other borings. The composition of the fill in this boring shows that it is similar in texture to the native loess soils. The applicant concluded that the fill appears to have been derived from the excavation of loess cut from areas of the GGNS site. As stated in its response, the applicant did not observe any evidence of settlement of the pavements or the filled ground surface during the ESP field work, suggesting that the fill is not unusually compressible or prone to settlement under its own weight or light pavement and vehicular loading. The applicant planned to place the foundations for the ESP reactor and safety-related facilities well below the maximum depth of these fills; therefore, the fill will not affect these facilities. The applicant also noted that the excavation to develop a uniform plant grade elevation in the ESP area would remove much of the fill underlying the upper pad area. However, a 10- to 30-foot-thick section of fill may remain under the eastern parts of the yard area. In addition, the applicant will take additional borings during the COL phase to evaluate the character of the fill and to determine if it will require additional excavations and replacements to minimize settlements of appurtenant nonsafety facility foundations and pavements.

To perform a probabilistic site response calculation, the base case is the best estimate (BE) velocity profile must be provided, together with the ±1 sigma shear wave velocity values (UB and LB values) over the entire soil profile. The staff asked the applicant in RAI 2.5.4-5 to provide the values of the BE, UB, and LB velocities selected for each primary component of the profile and the bases for its selection in either SSAR Section 2.5.4 or 2.5.2. In its response, the applicant stated that Figure 2.5.4-18 (SSAR Figure 2.5.4-60) shows the BE profile, which is based on a visual average of the three compression and shear (P-S) suspension log surveys obtained from the ESP site borings. The applicant's response to RAI 2.5.4-8 discusses the development of this BE profile. The applicant further noted that it did not develop the UB and LB profiles; instead, it used a profile randomization scheme to incorporate expected variability across the site. This approach is intended to maintain the 10^{-5} APE hazard level of the rock outcrop UHRS by developing statistically significant estimates of the mean amplification factors for the site.

The applicant stated that, of all the ESP borings, only one (Boring 2A) reached the depth of the Catahoula formation; however, even in this boring, no continuous core was taken inside the formation. The applicant noted that the UFSAR descriptions have improperly categorized the Old Alluvium of the Upland Complex as the Catahoula formation. With only one boring

available and no significant samples taken in this formation, the staff asked the applicant in RAI 2.5.4-1 to provide the basis for categorizing the Catahoula formation as bedrock as opposed to dense sands and gravels and the subsequent impact of this decision on the ESP site evaluation. In its response, the applicant compared the classifications from the UFSAR borings to those from the ESP investigation and correlated borings obtained during the two periods. The applicant noted that the nomenclature for the site geologic units used in the ESP differs from the convention used in the UFSAR. The stratigraphic nomenclature adopted for the UFSAR evaluations relies heavily on an older regional geologic study (USACE, "Geological Investigation of the Alluvial Valley of the Lower Mississippi River", 1944). To be consistent with the more recent regional geologic studies, the applicant updated and modified the soil nomenclature for the ESP site in the SSAR. These recent geologic studies provide a better understanding of the depositional history of Pliocene and Pleistocene sediments, as well as refined stratigraphic descriptions and correlations. The applicant correlated the UFSAR-defined Catahoula formation with the ESP-defined Upland Complex Old Alluvium. The applicant stated that the Old Alluvium differs from the underlying Catahoula formation by lesser degrees of lithification and coarser, less-sorted texture. Moreover, the Old Alluvium's contact with the overlying Upland Alluvium is an irregular erosion surface, and two of the borings show a yellowish-brown, slightly oxidized zone between the two. This oxidized zone helped the applicant establish the correlation between the ESP investigation and the description inside the UFSAR. The applicant also noted that the top of the Upland Complex Old Alluvium is in general about 30 to 80 feet higher in elevation at the GGNS site than underneath the ESP site. Therefore, foundations at the ESP site will require deeper excavations extending into the Upland Complex Old Alluvium to reach conditions that are similar to those of the existing plant. The existing plant foundations do not extend to the ESP-defined Catahoula formation, which consists of gray-green, hard clay to claystone that exhibits a slight degree of induration and somewhat brittle rock-like behavior. The descriptions for the ESP-defined Catahoula formation are similar to the indurated and/or partly cemented clays and silts (with some cemented sand lenses) that are described in the deeper parts of the UFSAR borings (i.e., below the bearing level of the plant foundation). The applicant believed that these materials are a better correlation to the regional descriptions of the Catahoula formation than the overlying, less-indurated sediments that are classified as Upland Complex Old Alluvium. On the basis discussed above, the applicant concluded that the change in nomenclature from that used in the UFSAR to that used in the ESP SSAR does not have any significant impact on the site evaluations because the properties for the equivalent cross-correlated units are essentially the same.

In SSAR Section 2.5.4.1.2, the applicant provided soil profiles developed for the ESP site, as shown in Figures 2.5.4-1 and 2.5.4-2. The maximum depth of these borings investigated during this program extends to about 240 feet. In RAI 2.5.4-3, the staff asked the applicant to provide the number and maximum depth of the borings that were used to characterize the ESP site area.

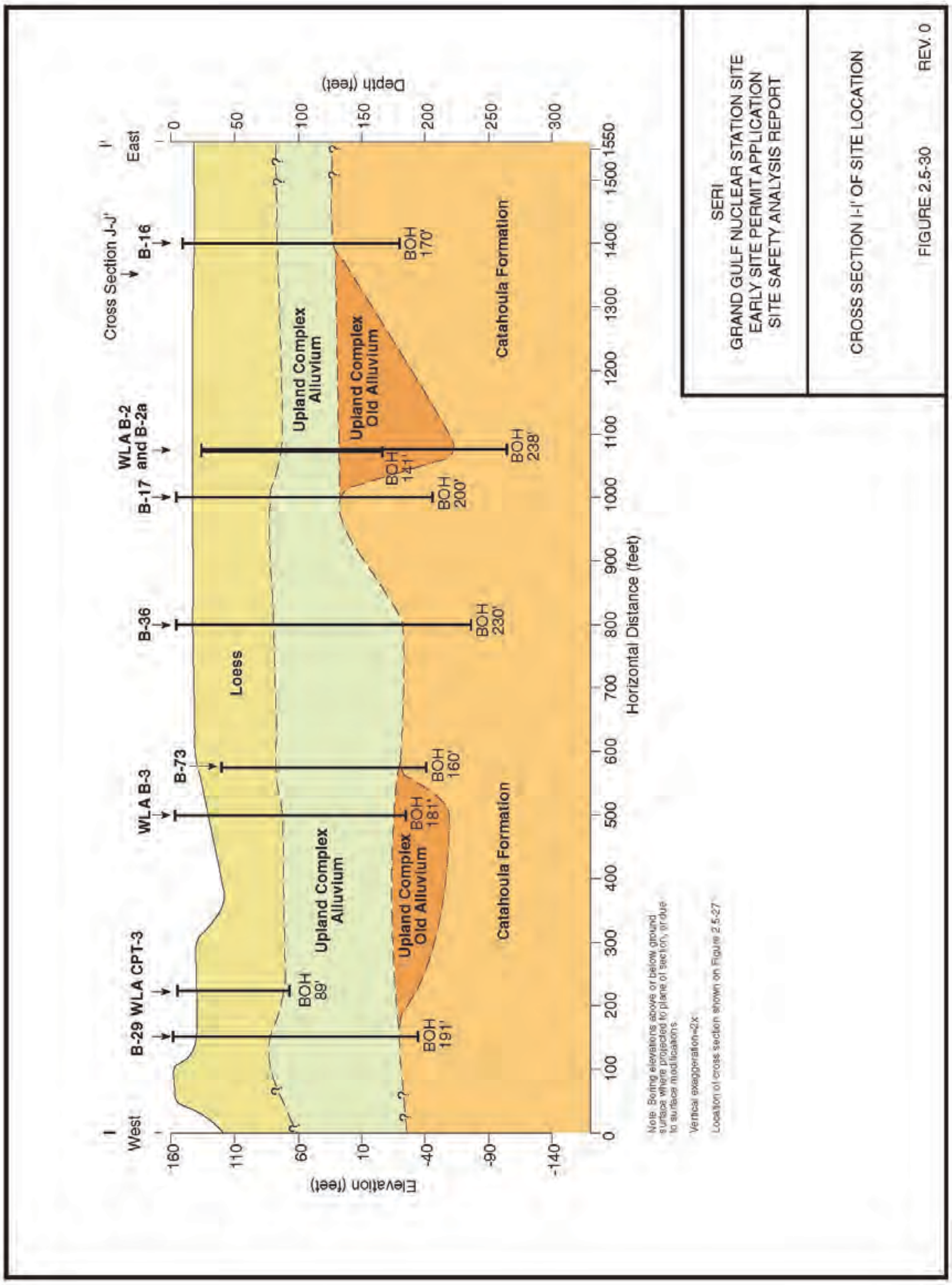


Figure 2.5.4-1 (SSAR Figure 2.5-30)
Cross Section I-I of Site Location

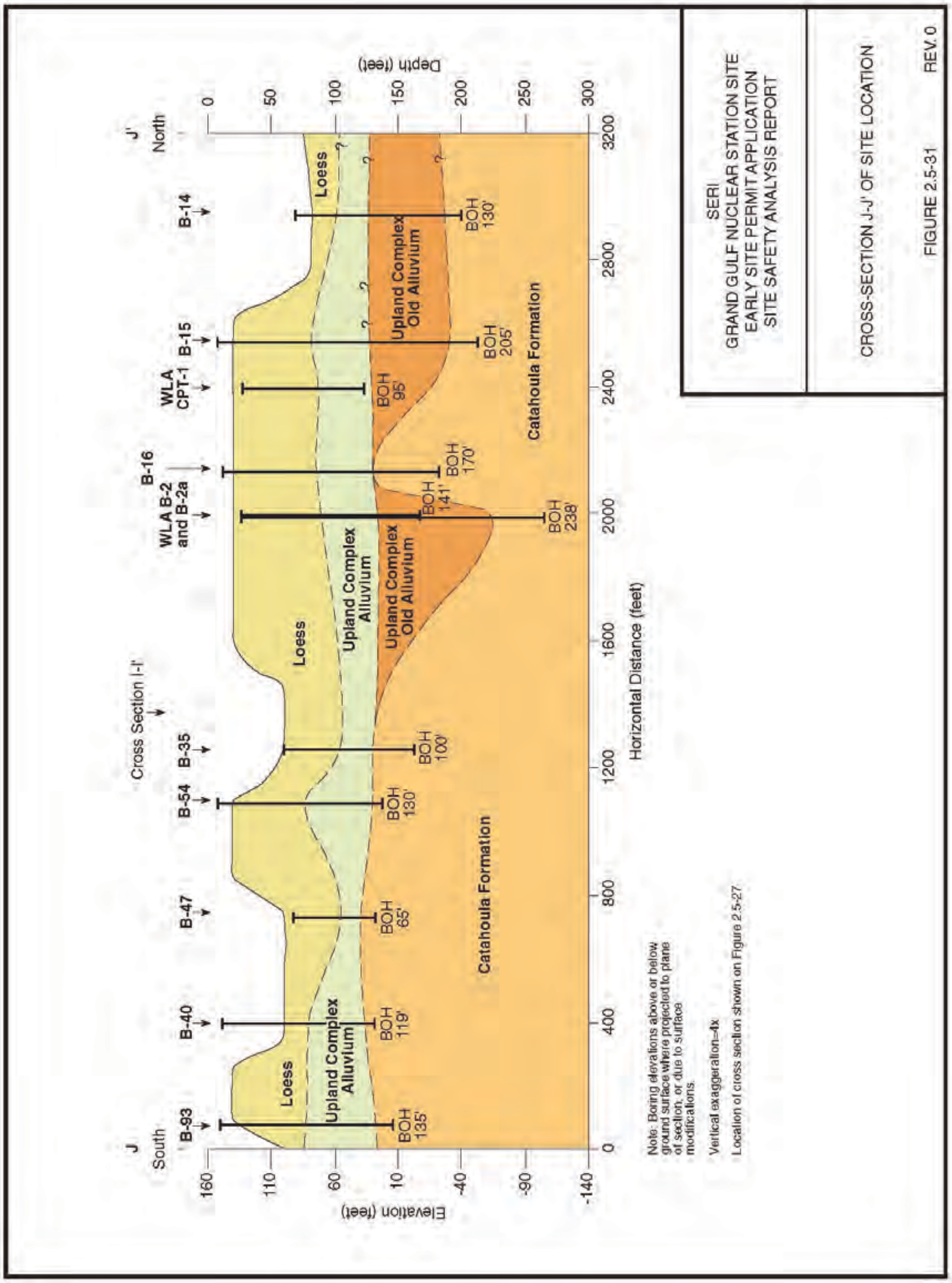


Figure 2.5.4-2 (SSAR Figure 2.5-31)
Cross Section J-J of Site Location

Presuming that these borings are also relatively shallow and that the site profile used in SSAR Section 2.5.2.3 extends thousands of feet deep into the hard rocks referred to in the EPRI-SOG ground motion attenuation relationships, the staff also asked the applicant to present additional information that allows for the characterization of the ESP site materials from a corresponding depth. In its response, the applicant explained that it characterized the ESP site by two individual borings, one composite boring, and four CPT soundings for the ESP site investigation program. The applicant distributed these borings and CPT soundings throughout the ESP site. The revised map (Figure 2.5.4-3) shows a circular PPBA that supersedes the previous polygonal proposed site location perimeter and indicates the approximate locations of the available UFSAR borings within the ESP area. The depths of the new ESP borings range between 142 and 238 feet below the ground surface while the depths of the CPT soundings range between 79 and 95 feet below the ground surface. The applicant augmented the ESP explorations with data from 20 previous borings performed for the UFSAR that fall within or adjacent to the ESP PPBA. The UFSAR borings extend to depths of between 81 and 300 feet below the ground surface. The combined coverage of the ESP and UFSAR borings provides sampling across the entire PPBA at 200- to 600-foot spacing and down to a depth of 300 feet. One of the ESP borings and about four to six of the UFSAR borings extend into the dense clay to claystone that the applicant defined as the Miocene Catahoula formation. Two of these borings include interval core sampling of the Catahoula formation. Revised SSAR Figure 2.5-69 integrates the ESP and the UFSAR explorations and indicates a good control to a depth of about 250 feet below the ground surface.

The applicant stated that it developed the ESP site investigation program to obtain sufficient information to characterize the site's subsurface conditions because soil variability may influence the earthquake ground motion response analysis. The ESP site explorations, augmented by borehole data from the UFSAR, capture the three-dimensional geometry of the strata and variations in soil properties. The applicant also stated that the generally consistent horizontal stratigraphy of the site provides a high level of confidence regarding the characterization of the geologic and geotechnical conditions appropriate for the ESP study. The applicant will further verify the site stratigraphy by additional borings taken during the COL phase. After it reviewed the data from a series of cross-hole seismic surveys performed for the UFSAR in four borings at the existing power plant area, the applicant obtained the cross-hole wave velocities to a depth of 300 feet and recorded wave velocities at 10-foot intervals to develop a site velocity profile. However, the applicant stated that the cross-hole survey used explosives, a poorly controlled source, set off in a fifth borehole, as well as old approaches and equipment that cannot be reliably correlated to the ESP site borehole seismic P-S surveys. Therefore, the applicant only used the UFSAR cross-hole velocity profiles as a general comparison to the ESP site-specific data, and the comparison did not provide information useful to extrapolate the ESP profiles to greater depth. The applicant extended the velocity profile below a depth of 300 feet using a generic deep profile developed from regional studies of the Mississippi embayment. The applicant also noted that these regional studies include extensive velocity measurements near the Memphis area and its surrounding regions, as well as the development of regional soil columns for the Mississippi lowlands and uplands. The generic profile accounts for the depth to the Paleozoic crystalline basement for long-period responses. The applicant's response to RAI 2.5.4-4 discusses this extrapolation in detail.

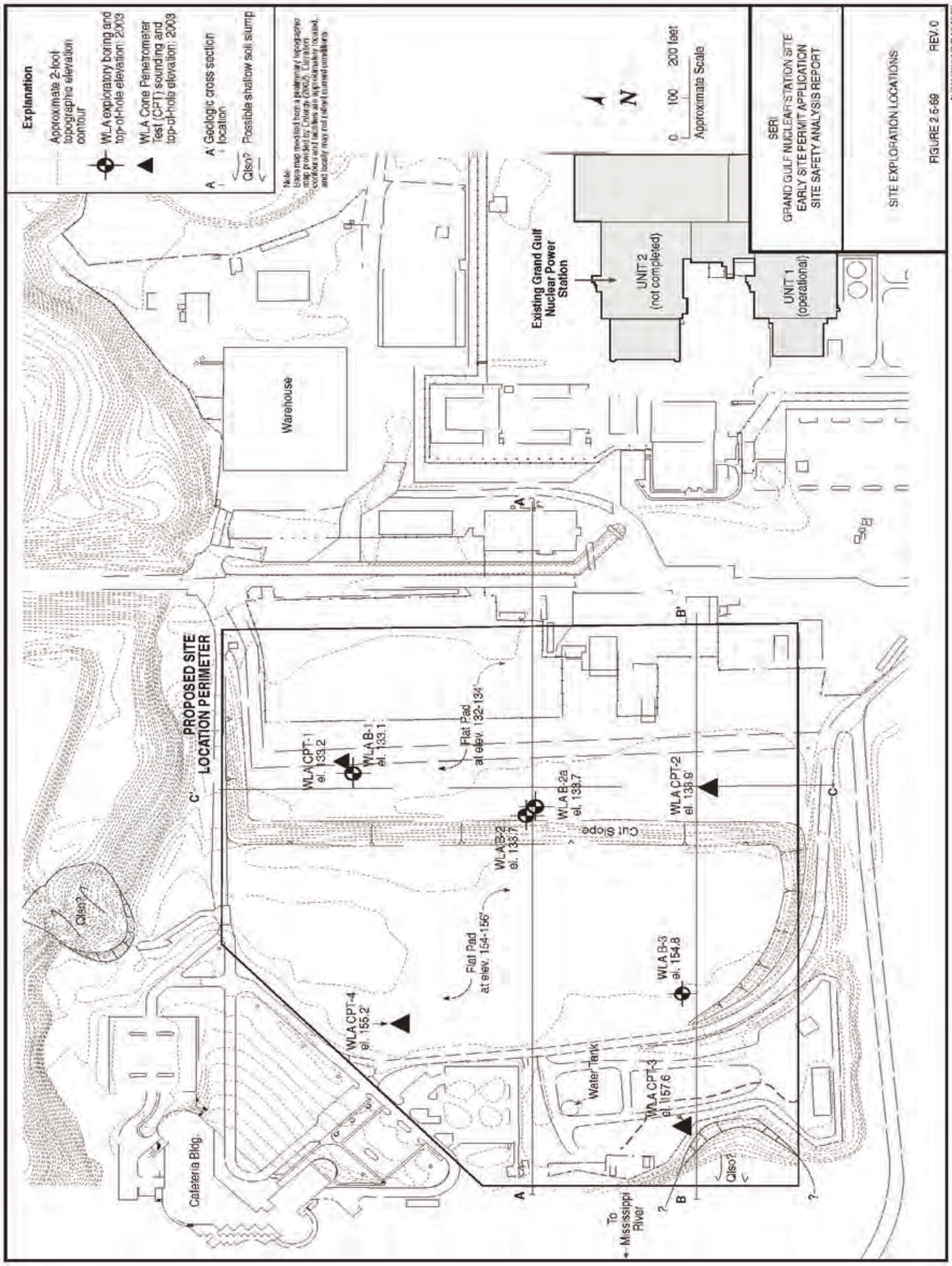


Figure 2.5.4-3 (Revision 2 of SSAR Figure 2.5-69)
Site Exploration Locations

Geophysical Properties. As stated in SSAR Sections 2.5.4.1.3 and 2.5.4.1.4, the applicant performed suspension logging using an OYO logging system that measures both compression (P) and horizontal shear (S) wave velocities in the three borings drilled in the ESP site area. Figure 2.5.4-4 presents the results of the logger measurements. The results indicate relatively slowly increasing S-wave velocities over the depths investigated in the boreholes. At a depth of about 200 feet below the ground surface, the S-wave velocity is about 2000 fps and the corresponding P-wave velocity is about 6000 to 7000 fps. The P-wave velocity increases slowly with depth, except in the vicinity of the ground water table where it increases relatively rapidly as the degree of saturation increases. No other significant difference in profile velocity occurs at any anticipated interface between the layers described above. Based on previous information obtained from the UFSAR for the existing GGNS, the applicant noted that the velocities obtained from cross-hole logging are in the same value range.

Cone Penetrometer Testing. As stated in SSAR Section 2.5.4.1.5, the applicant pressed four CPT soundings across the site to depths between 79 and 95.3 feet through the loess and into the Upland Alluvium. Resistance is relatively consistent (and relatively low) through the loess and increases significantly into the Upland Alluvium. Correlation of strength and grain-size properties using standard correlations indicates that the CPT results are generally consistent with sample SPT blow counts. Gradations are generally consistent with those found from inspection of the samples. The CPT data indicate that the loess and Upland Alluvium are somewhat overconsolidated.

Static Laboratory Testing. SSAR Section 2.5.4.1.6 summarizes the laboratory data obtained from 60 samples of site soils obtained from the ESP investigation. The majority of static tests conducted were for standard index properties, such as moisture content, dry density, Atterberg Limits, and grain-size distribution (sieve shaking and hydrometer). The applicant conducted six consolidated-undrained triaxial test series (consisting of from one to three consolidation pressures in each test series) to obtain static strength parameters. Tables 2.5-24 and 2.5-25 of the SSAR summarize those results.

The results indicate that the loess has an average dry density of 1522.18 kilograms per cubic meter (kg/m^3) (94.8 pcf) with an average moisture content of 22 percent. The Upland Alluvium has an average dry density of 1666.39 kg/m^3 (106 pcf) with an average moisture content of 68 percent. The Old Alluvium has an average density of 1522.18 kg/m^3 (94.7 pcf) with a moisture content of 23 percent. This density of the Old Alluvium is unusually low and is similar to that of the loess. Only one sample of the Catahoula formation was available for testing, and no density or strength information was obtained from this sample. The applicant noted that theresults obtained from these laboratory tests are similar to those indicated in the UFSAR for the loess.

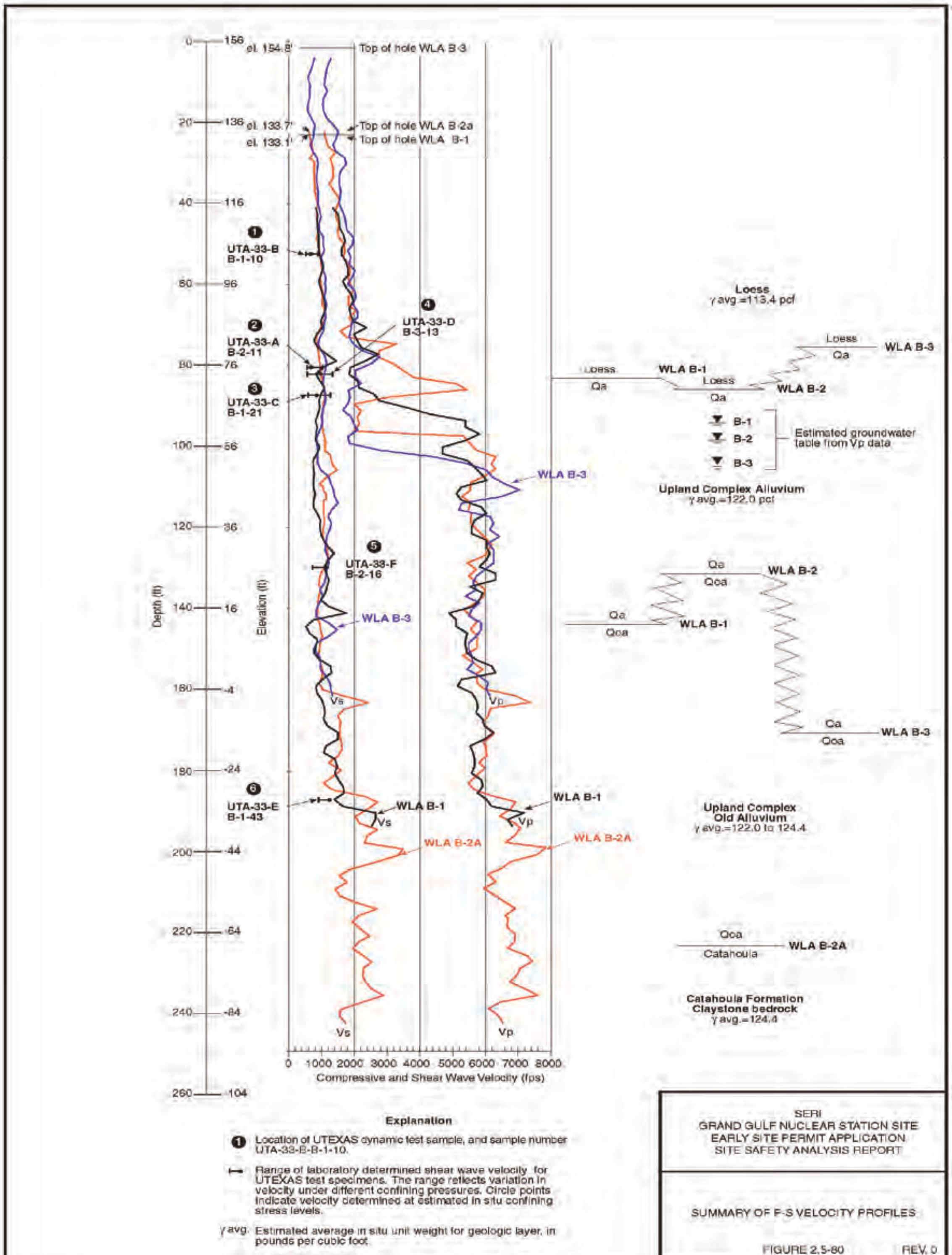


Figure 2.5.4-4 (Figure 2.5-80 of the SSAR)
Summary of P-S Velocity Profiles

Dynamic Laboratory Testing. As described in SSAR Section 2.5.4.1.7, the applicant performed laboratory tests on six samples to obtain the dynamic material properties needed to perform the site response evaluations used in developing design spectra. In addition to the standard index testing normally performed on soil samples, the applicant obtained the dynamic properties from resonant column (RC) and torsional shear (TS) testing. Of the six samples tested, two each are from the loess, the Upland Alluvium, and the Old Alluvium, respectively. As indicated in Section 11.0 of ER-02, these zones represent the effective depth over which undisturbed samples can be pushed using ordinary thin-walled Shelby tube samplers. The test results obtained were the low-strain values of shear wave velocity and hysteretic damping ratio together with shear modulus reduction and hysteretic damping curves as functions of peak effective shear strain developed in the material. Low-strain values of the parameters are typically defined at a peak effective shear strain of 10^{-4} percent. Modulus reduction and damping data are typically desired over a shear strain range of 10^{-4} to 1 percent effective strain, depending on the level of strain anticipated in the site soil column for the given level of seismic shaking resulting from the design ground motion.

SSAR Table 2.5-26 outlines the results of the testing program. A comparison of the low-strain shear wave velocities measured in the laboratory with the values measured during the field geophysical program indicates a ratio of laboratory to field velocities varying from as low as 0.58 to as high as 1.00. Five of the six samples show a ratio of laboratory to field velocities significantly below unity. As described in Section 11.0 of ER-02, the fact that the ratio is less than unity is typically ascribed to the disturbance of the soil sample caused by the sampling process and the disturbance induced during the testing program.

In the laboratory, the applicant confined the samples at a specified value of confining pressure defined by the stress ratio parameter k_0. For a given value of overburden vertical effective stress (defined typically by the total weight of material above the sample depth minus the effects of ground water), the parameter k_0 represents the ratio of the corresponding value of lateral effective stress to the value of vertical effective stress. This ratio is generally unknown in situ but is anticipated to range from a value of 0.5 to 1.0, depending upon the conditions at the site, method of deposition, and soil type, among other factors. The applicant used a value of 0.5 for the loess samples and a value of 1.0 for the Upland Old Alluvium samples. After the completion of the initial testing program, the applicant tested the samples at confining pressures of 4 times the estimated in situ mean effective pressures. The applicant stated that the stress ratio parameter k_0 used for the loess samples was most likely too low and the value of unity was probably more appropriate.

The applicant compared the strain-dependent shear modulus reduction and damping curves generated from the test program with the family of curves recommended in EPRI TR-102293, “Guidelines for Determining Design Basis Ground Motions,” issued November 1993 (EPRI-TR) and concluded that the test results are reasonably consistent with the curves recommended in the EPRI report. However, the applicant observed that the test results are more linear than the EPRI curves at the comparable depth of embedment (or confining pressure) for the sample. The applicant also noted that a possible reason for this is that the EPRI curves are reasonably appropriate for relatively young and normally consolidated Holocene soils, while the ESP site soils are generally older and somewhat overconsolidated. The applicant then evaluated the test results to generate appropriate functions and used these test results for the site soils modeled in the site response analyses.

In SSAR Section 2.5.4.1.7 and ER-02, the applicant stated that the site response calculations used the EPRI-TR depth-dependent curves as the shear modulus reduction and hysteretic damping models. The staff considers these to be generally appropriate for normally consolidated cohesionless sands. As described in ER-02 Sections 11.0 and 12.0, these curves may not be appropriate for the near-surface layers of the soil profile for which laboratory data are available; further, they may not be appropriate for any gravelly layer in the profile that tends to behave significantly more nonlinear than those indicated by the EPRI-TR dataset. In RAI 2.5.4-6, the staff asked the applicant to provide its basis for selecting the EPRI-TR curves as opposed to other models that may be more appropriate based on site-specific information described in the geotechnical report. In its response, the applicant noted that the statement that the shear modulus reduction and hysteretic damping curves used in the site response calculations are the EPRI-TR depth-dependent curves is correct but not complete. The SSAR description does not specify which of the EPRI curves are to be used for the various units in the site response analyses. SSAR Section 2.5.2.3 includes specifications for assigning the EPRI curves to the three principal units involved in the site response analyses (loess, Upland Alluvium, and Old Alluvium). The applicant did not use the EPRI curves corresponding to the depth ranges for these three soil units; rather, it used EPRI curves that represent greater depths of normally consolidated soils to represent the aged and overconsolidated soils at the ESP site. For example, the applicant used the EPRI-TR curves for depths of 120 to 250 feet to represent the loess, the EPRI-TR curves for depths of 250 to 500 feet to represent the Upland Alluvium, and the EPRI-TR curves for depths of 500 to 1000 feet to represent the Old Alluvium and the underlying materials to a depth of 500 feet. Below a depth of 500 feet, the applicant assumed that the profile exhibits essentially linear material properties.

The applicant also stated in its response that SSAR Section 2.5.4.1.7 presents the results of the dynamic laboratory tests plotted on the families of EPRI modulus reduction and damping curves for loess, Upland Alluvium, and Old Alluvium. However, because it did not indicate which of the EPRI curves should be selected for the site response analyses, the applicant will revise these SSAR figures to indicate the recommended curves used in the site response analyses. The applicant also stated that SSAR Section 2.5.4.1.7 discusses the rationale for adopting the EPRI curves. Specifically, the shape of the curves defined by the site-specific laboratory tests is similar to the shape of the EPRI curves. The comparison of the laboratory and EPRI-TR curves indicates that the raw laboratory data show less modulus reduction and lower damping values than the EPRI curves for the same depths. The applicant also explained that this is likely because EPRI developed these curves for normally consolidated Holocene silty and clayey sands, whereas the soils at the ESP site are both older and overconsolidated. In addition, the applicant noted that the laboratory test results must be adjusted for the effects of stress relief sample disturbance. As indicated in SSAR Table 2.5-26, the ratio of the shear wave velocities measured in the laboratory versus the field is generally less than unity. The ratio of the shear wave velocities measured in the laboratory for specimens consolidated to 4 times the estimated in situ stress versus the field-measured velocities are on the order of unity. This suggests that the additional sample consolidation produces results that better match the in situ field conditions of these soils.

In addition, the applicant stated in its response that RAI 2.5.4-6 correctly points out that the EPRI depth-dependent curves may not be appropriate for gravels. The EPRI report includes generic curves for pure gravels that are relatively more nonlinear than the family of depth-dependent curves that apply to soils that range from gravelly sands to low plasticity silty or

clayey sands. However, at the ESP site, the gravel layers consist of some gravel-size particles in a sandier matrix. Such gravelly materials are generally less than 5-feet thick and appear to be discontinuous. Therefore, the applicant noted that the use of the EPRI-TR gravel curves would not be appropriate for the ESP site conditions.

In SSAR Section 2.5.4.1.7, together with ER-02 Section 11.0 and Appendix G to ER-02, the applicant presented a detailed summary of the laboratory dynamic test results and provided a comparison of the shear modulus reduction and hysteretic damping curves generated from the laboratory testing with the EPRI-TR recommendations used in the site response calculations.

As indicated in SSAR Section 2.5.2.3, the applicant used the EPRI-TR model for all layers of the soil profile. The staff identified the following issues in RAI 2.5.4-7 as a result of its review of the EPRI-TR:

- The shear modulus data presented in Appendix G for samples taken from a shallow depth are reasonably comparable to the EPRI recommendations for depths of 500 to 1000 feet. The hysteretic damping ratios from the laboratory tests are also much lower than those indicated in the EPRI-TR recommendations for comparable sample depths. Therefore, the laboratory data indicate properties that are much more linear and possess lower damping than those represented by the EPRI-TR recommendations for similar sample depths.

- The boring logs reported in Appendix C to ER-02 indicate that some soil layers have significant gravel content. These materials may normally be expected to have properties that are much more nonlinear than indicated by the EPRI-TR recommendations.

- Section 11.0 of ER-02 suggests that the laboratory results for specimens UTEXAS 1, 2, and 6 should be corrected to account for the effects of sample disturbance and/or underestimation of effective confining stress.

In RAI 2.5.4-7, the staff asked the applicant to provide its rationale for not incorporating these effects in the site response calculations and to evaluate the potential impact of these modifications on the computed surface UHRS. In its response, the applicant referred to its response to RAI 2.5.4-6 and stated that the analysis and selection of the appropriate EPRI-TR curves for shear modulus reduction and hysteretic damping fully account for the effects of possible disturbance to the test specimens resulting from stress release during sampling, as well as possible movements during testing. The applicant considered the effects of sample relaxation by examining the results from testing soils at both the estimated in situ confining stress and 4 times the estimated in situ confining stress. The applicant plotted these two data sets together on the EPRI-TR base curves as shown in Figures 2.5.4-5 through 2.5.4-12, which replicate SSAR Figures 2.5.4-87 to 2.5.4-94. The two data sets were weighted to select the most representative EPRI-TR curve.

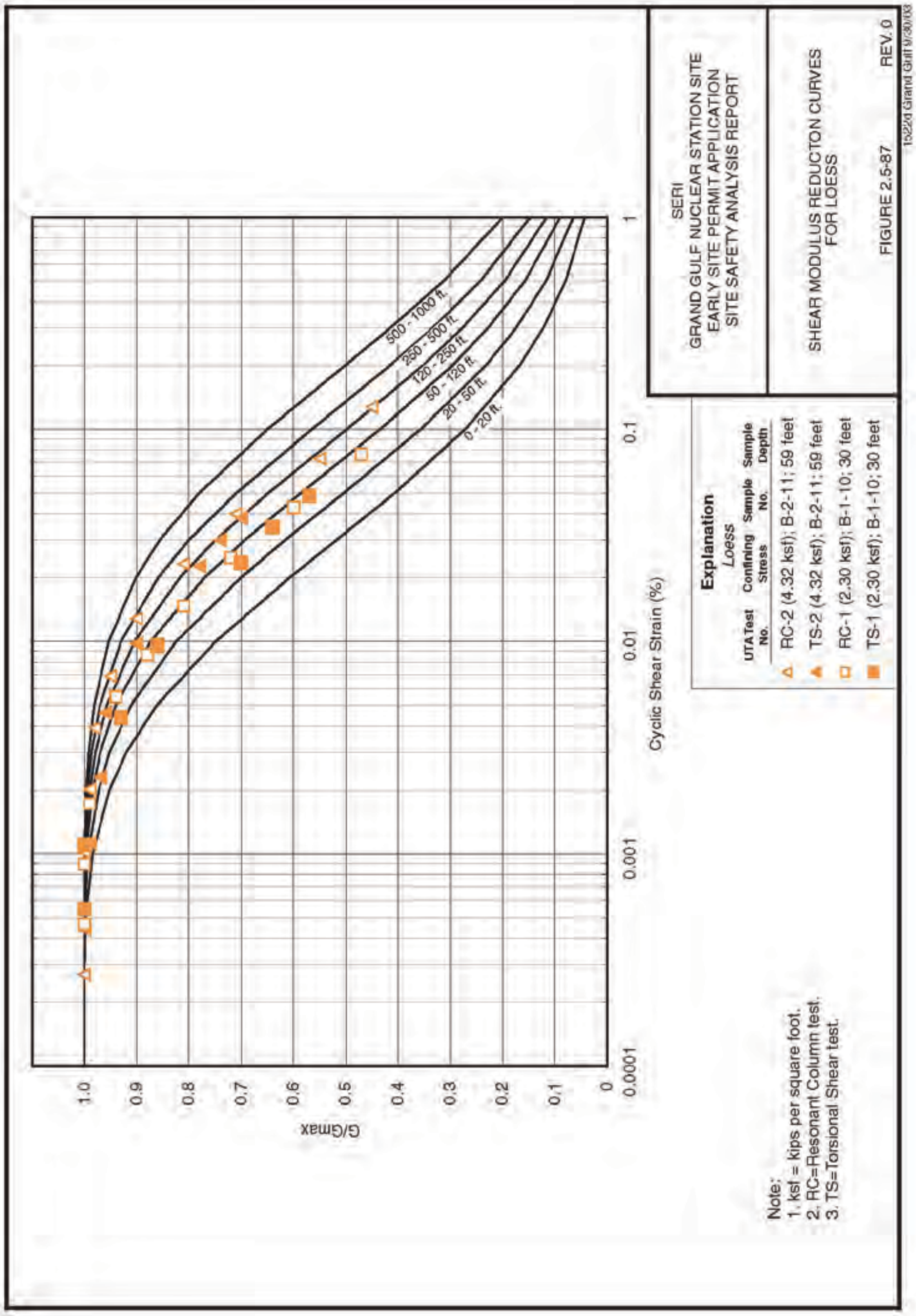


Figure 2.5.4-5 (Updated final version of SSAR Figure 2.5-87)
Shear Modulus Reduction Curves For Loess

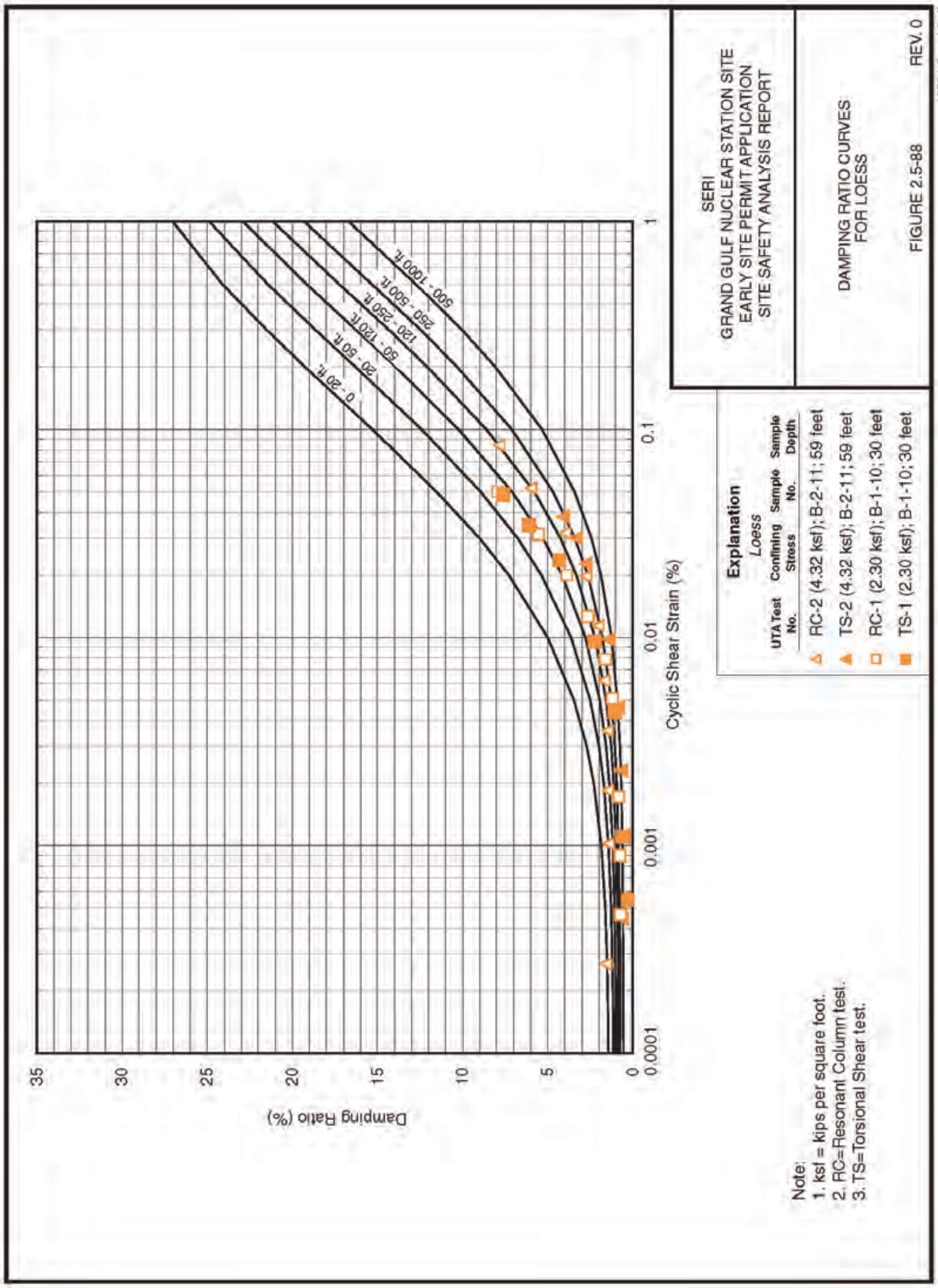


Figure 2.5.4-6 (Updated final version of SSAR Figure 2.5.4-88) Damping Ratio Curves for Loess

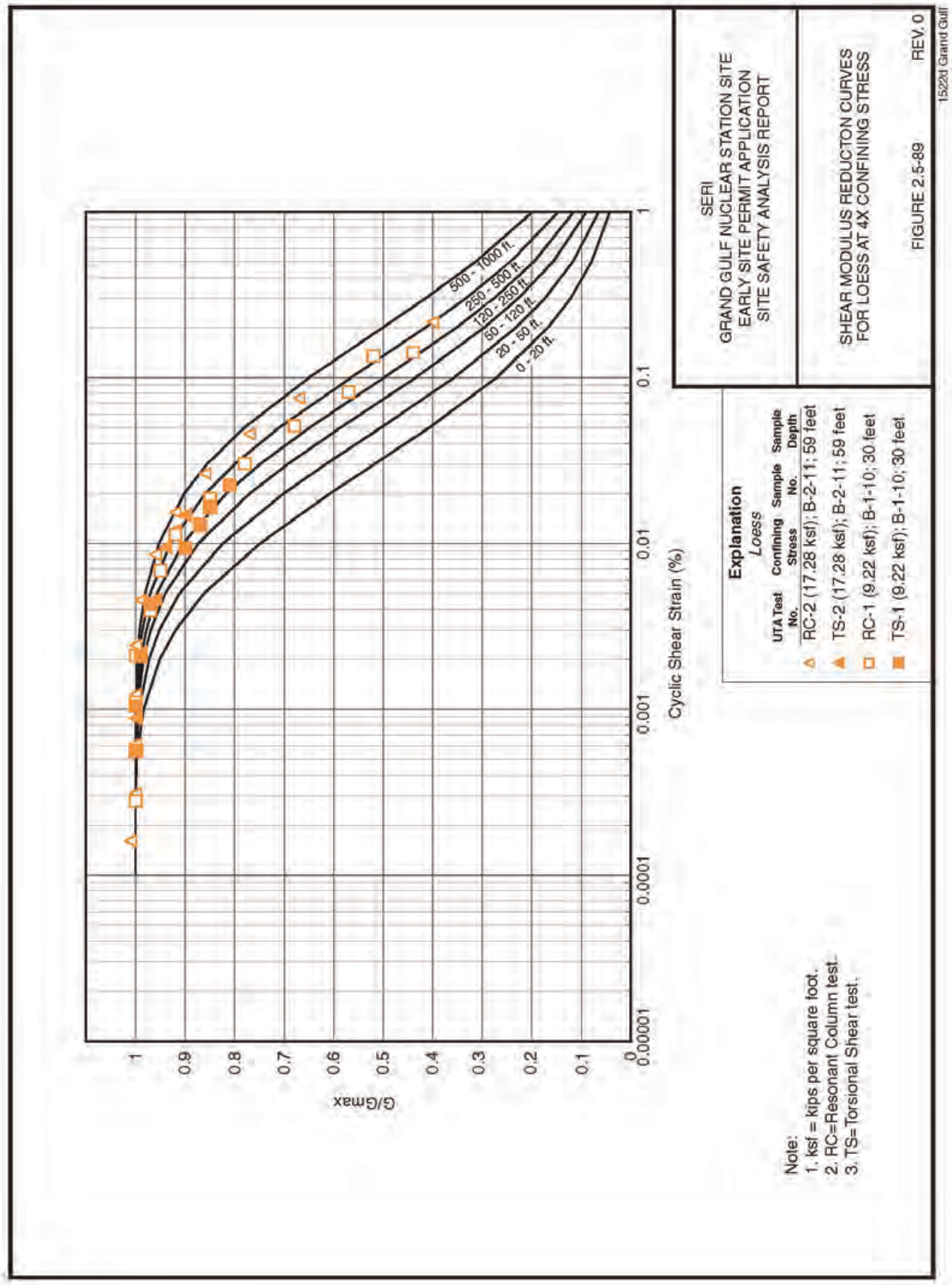


Figure 2.5.4-7 (Updated final version of SSAR Figure 2.5.4-89)
Shear Modulus Reduction Curves for Loess at 4X Confining Stress

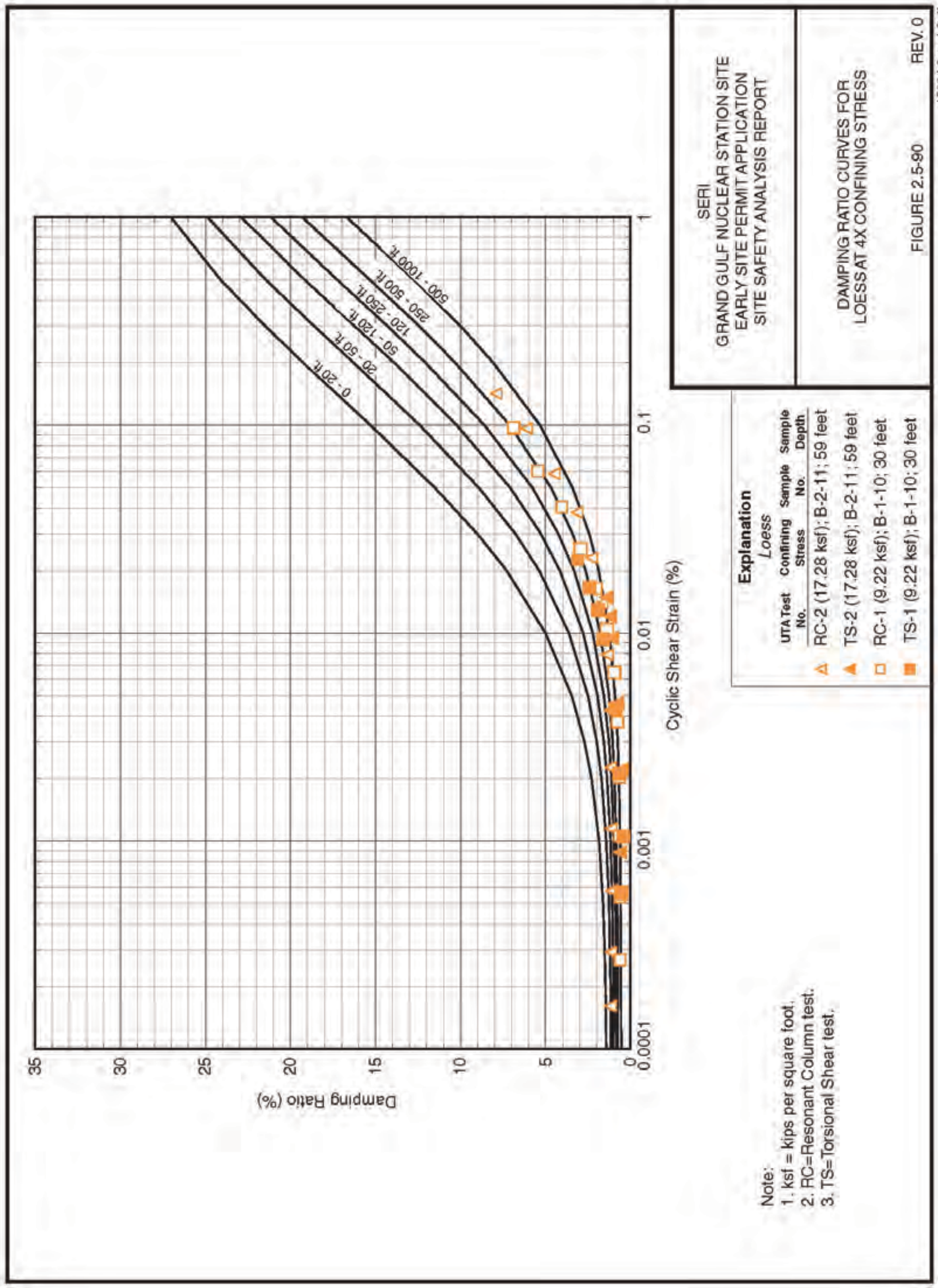


Figure 2.5.4-8 (Updated final version of SSAR Figure 2.5.4-90)
Damping Ratio Curves for Loess at 4X Confining Stress

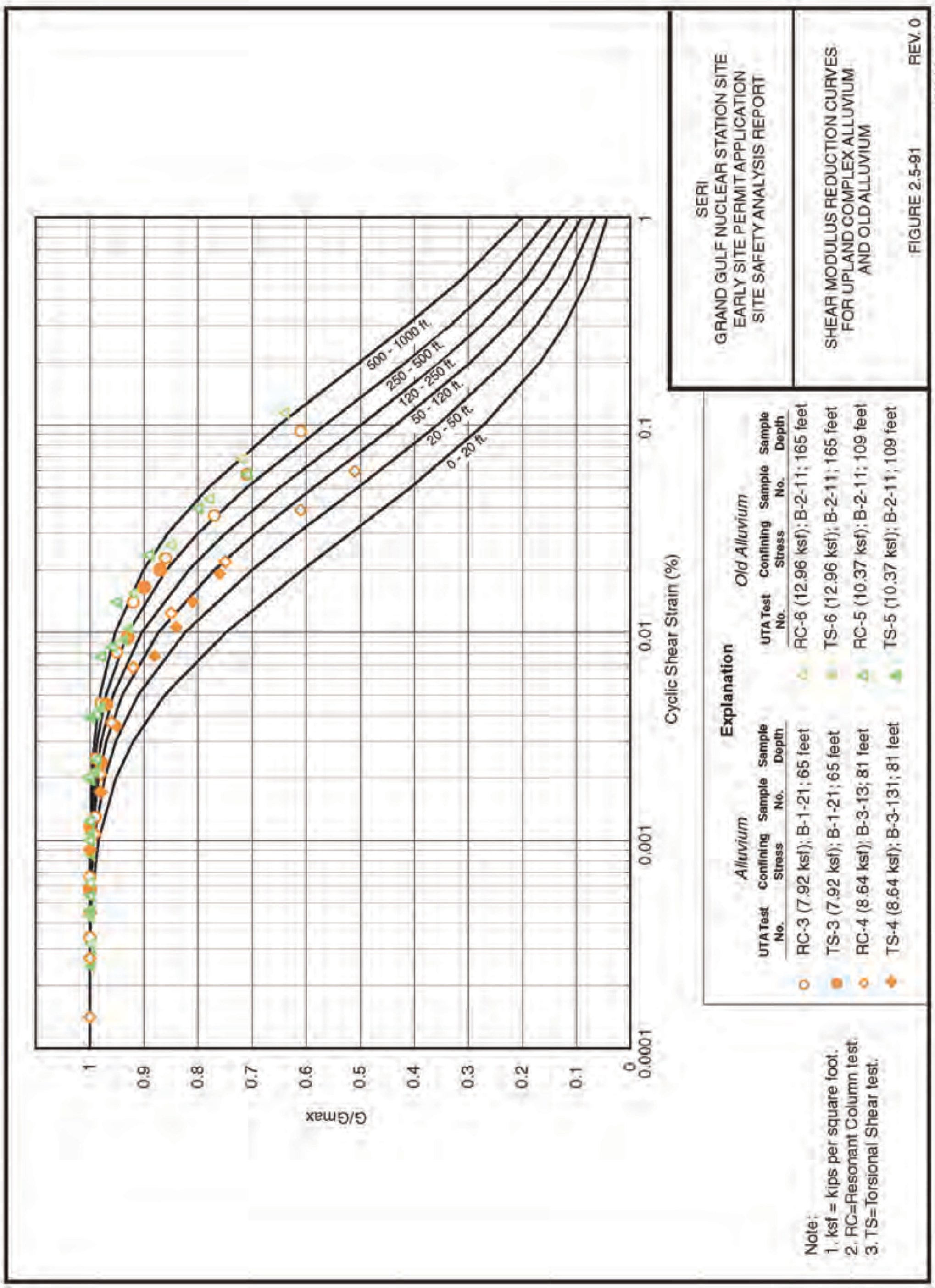


Figure 2.5.4-9 (Updated final version of SSAR Figure 2.5.4-91)
Shear Modulus Reduction Curves for Upland Complex Alluvium and Old Alluvium

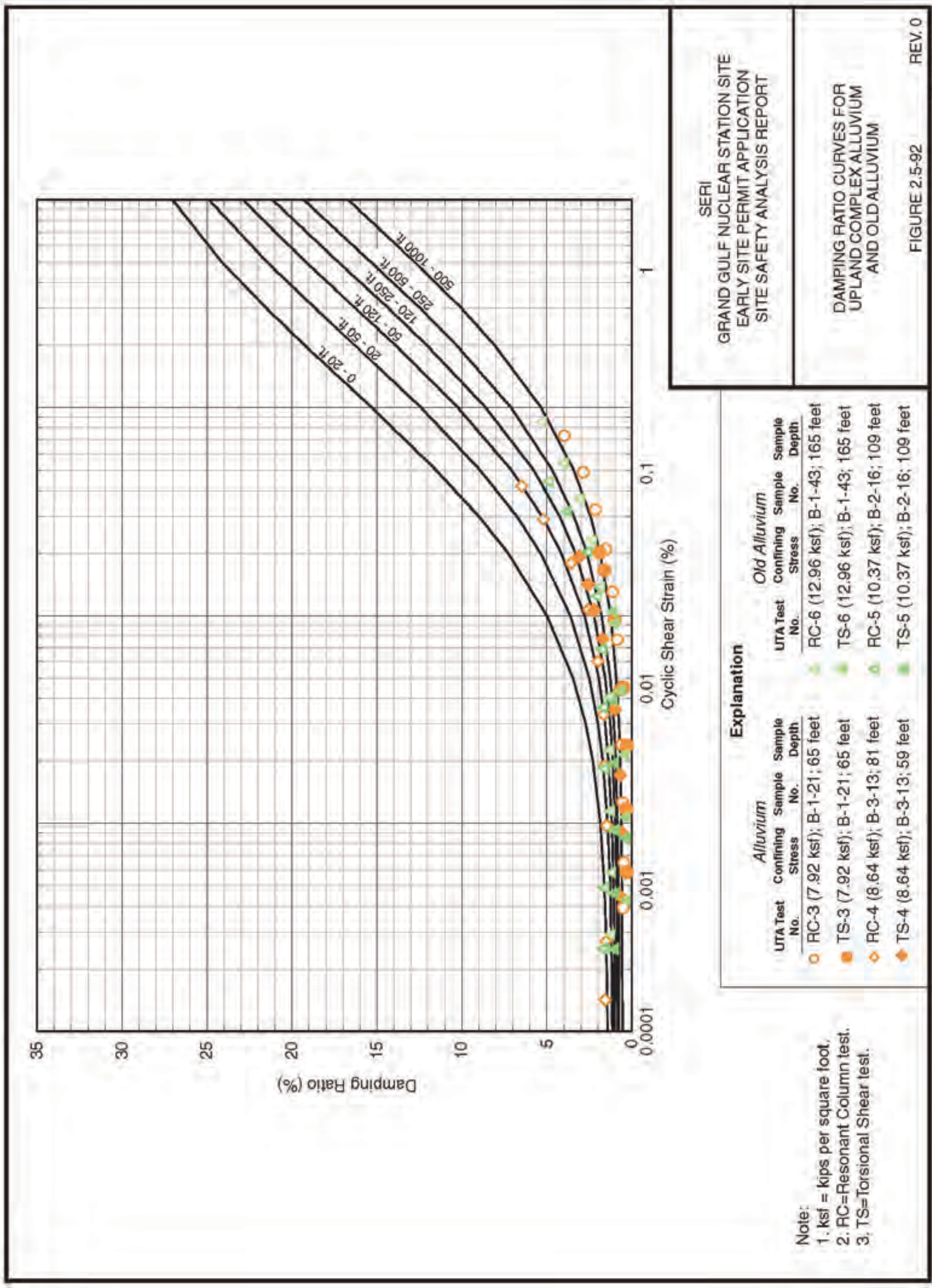


Figure 2.5.4-10 (Updated final version of SSAR Figure 2.5.4-92)
Damping Ratio Curves for Upland Complex Alluvium
and Old Alluvium

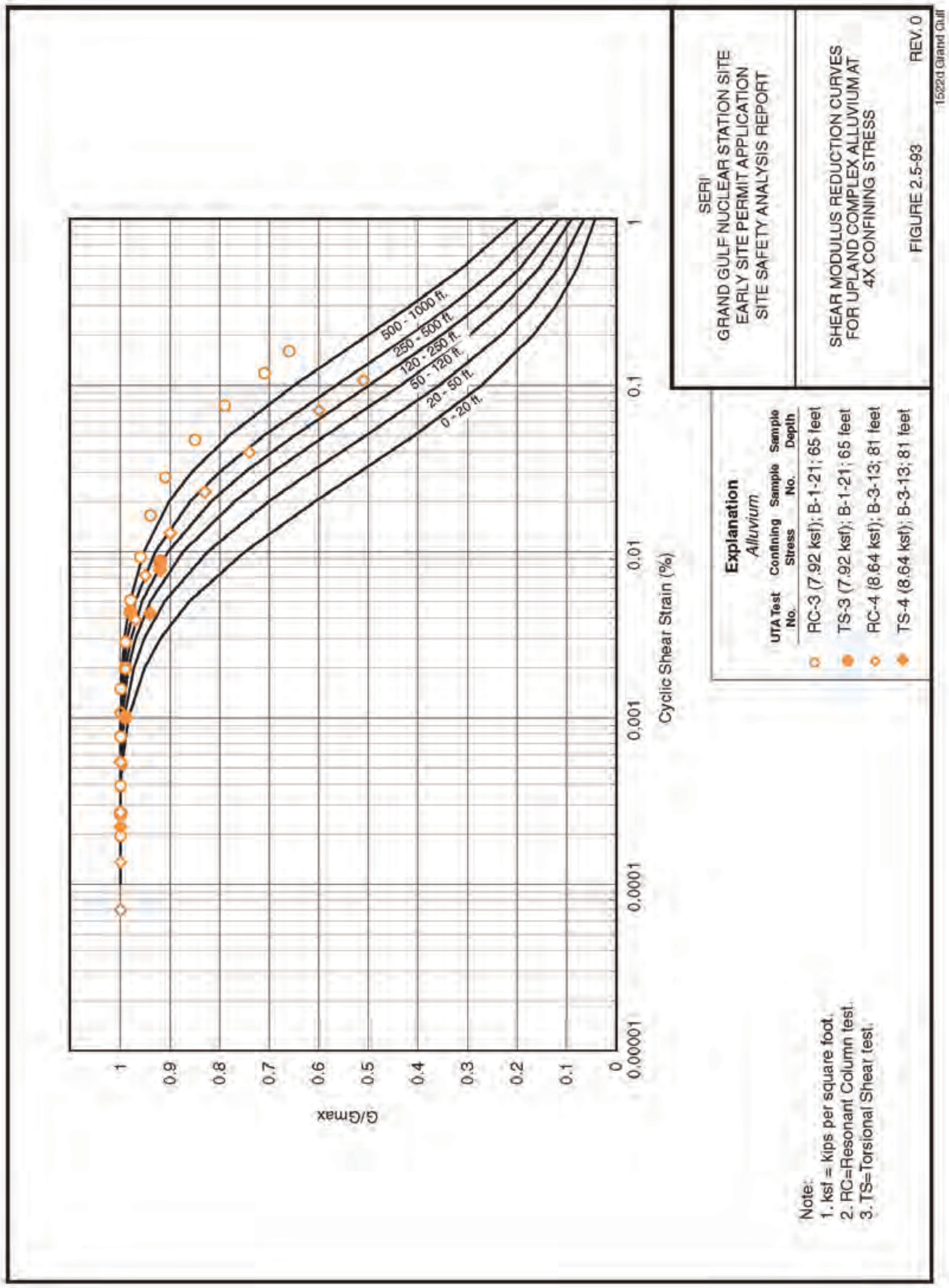


Figure 2.5.4-11 (Updated final version of SSAR Figure 2.5.4-93) Shear Modulus Reduction Curves for Upland Complex Alluvium at 4X Confining Stress

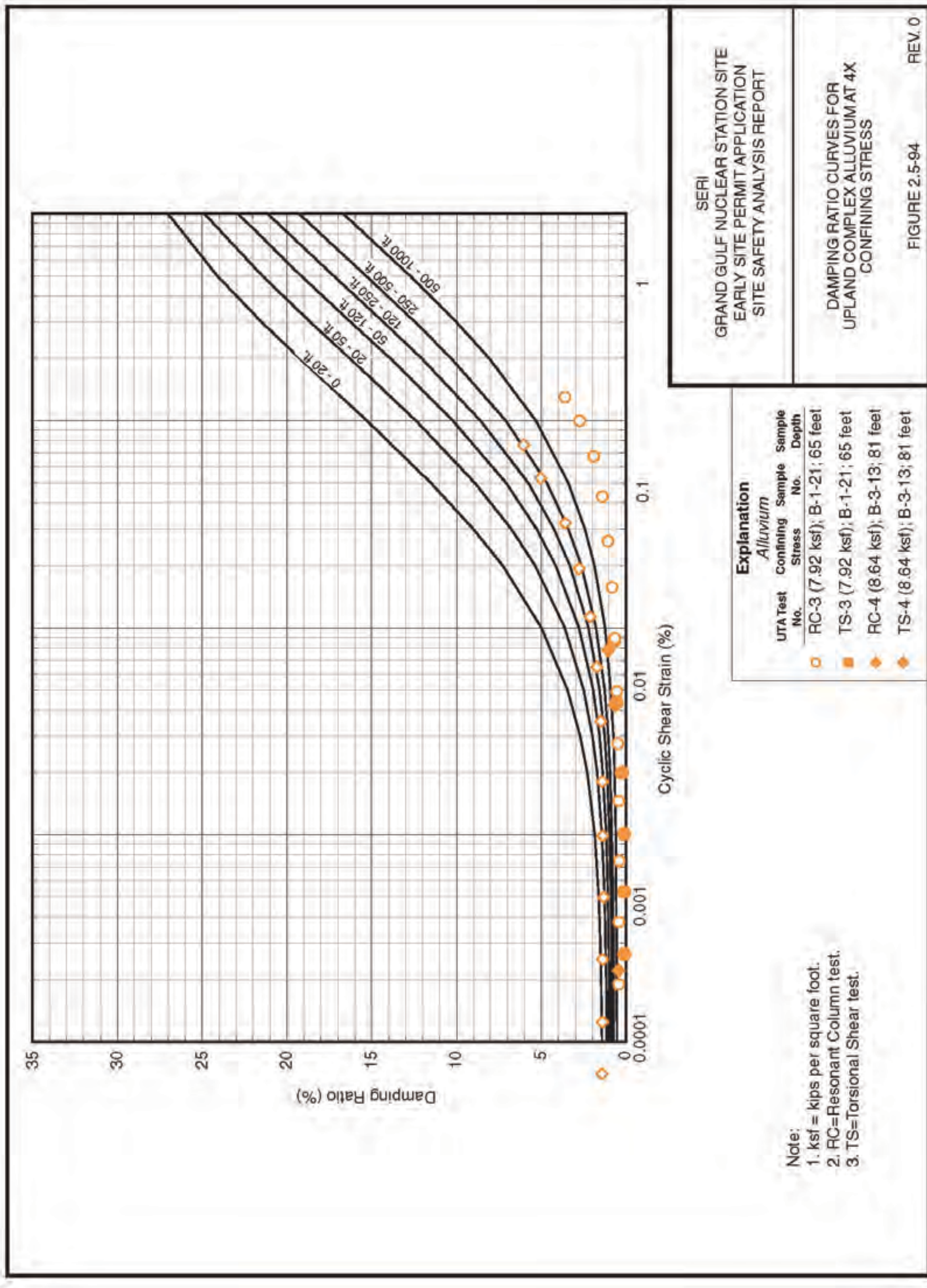


Figure 2.5.4-12 (Updated final version of SSAR Figure 2.5.4-94)
Damping Ratio Curves for Upland Complex Alluvium at
4X Confining Stress

In its response, the applicant further stated that the shear modulus reduction and damping curves adopted for use in the site response analysis represent the best fit to the dynamic soil test results that account for the effects of soil disturbance and overconsolidation. The applicant also considered the uncertainty and variability in these values in the site response modeling by developing randomizing curves about the selected base case EPRI-TR curves using an assumed lognormal distribution. The applicant further discussed the randomization process in its response to RAI 2.5.4-8 (for shear wave velocity input). The applicant considered that the EPRI-TR curves selected for the base case for site response analysis are appropriate for the site soil conditions and account for both sample disturbance and overconsolidation. Therefore, incorporation of these issues would not impact the computed surface UHRS.

Engineering Properties. The applicant summarized in ER-02 the engineering properties of the subsurface materials derived from the ESP field exploration and laboratory testing programs. These properties consist of data typically used in designing heavy foundations subjected to static loads, including natural moisture content, consolidation characteristics, undrained shear strength, effective cohesion, effective friction angle, total unit weight, SPT values, and static earth pressure coefficients. In a design, these properties must be defined or bounded for each individual zone in the profile that is expected to be directly influenced by the foundation loads. The applicant would likely place the foundations of any new Category I facilities at the depth of the Catahoula formation because the surface soils are relatively soft. Therefore, the applicant must only determine the soil property to the depth of the Catahoula formation that is relatively hard and capable of providing adequate support. The data developed during the ESP program, therefore, focus on the loess, the Upland Alluvium, and the Old Alluvium that overlie the Catahoula formation.

In the site response calculations performed to generate the soil design response spectrum, information on low-strain shear wave velocity and hysteretic damping ratio is required for all the soils in the profile, together with the variability from the BE values. No information is available below the upper 197 feet of the profile. Even for the upper 197 feet of the profile, few data are available on which to estimate appropriate UB and LB values of the soil properties. The applicant did not present any information on the values assumed for the site response calculations.

In seismic response analyses, shear and compression wave velocities, material densities, and strain-dependent modulus reduction and hysteretic damping data are necessary for the entire soil column extending to the depth considered in the site response evaluation. As indicated in Figure 2.5.2-4, the applicant assumed that the hard rock, with a shear wave velocity of 9186 fps, is located at a depth of 3281 feet and adopted a generic profile proposed for the Mississippi embayment region. This shear wave profile is relatively smooth, with a shear wave velocity varying from approximately 820 fps at the ground surface to a value of 9186 fps at a depth of 3281 feet. The applicant also replaced the upper 197 feet of the profile with a site-specific velocity profile generated by the geophysical program described previously. Therefore, of the entire 3281 feet of the profile, only the upper 197 feet are reasonably constrained. No site-specific information is available below this depth. However, it is a common practice for the sites where critical facilities are located to have one or more base case shear wave velocity models from site-specific data (e.g., well logs and deep borings) and then to generate soil UHRS using the probabilistic method of NUREG/CR-6728. In RAI 2.5.4-4, the

staff asked the applicant to provide the basis for selecting the generic base case velocity model as opposed to any other model that might be generated from available information for the site and its environs. In its response, the applicant stated that no measurements of velocity profiles are known to exist within tens of kilometers of the site. Based on extensive measurements near the Memphis area and surrounding regions, the differences in stratigraphy and wave velocities between the lowlands and uplands extend only to shallow depths (upper several hundred feet), are reasonably well characterized, and are generally considered uniform throughout the embayment.

A number of surface wave analyses as well as low-frequency site amplification studies also suggest generally uniform soil column properties throughout the embayment. The depth to the Paleozoic basement is greatest along the axis of the Mississippi River near the center of the embayment and diminishes to zero northward near Cairo, Illinois, as well as in easterly and westerly directions from the river. The general lateral uniformity of the deeper wave velocities (deeper than several hundred feet) is consistent with other large basins (e.g., the Los Angeles basin and Imperial Valley, California) and provides a basis for employing a generic deep profile beneath the local site profile. Since the profile is randomized, expected fluctuations are accommodated in the mean amplification, which is consistent with a PSHA. Additionally, differences in a mean shear wave velocity profile gradient, apart from the measurement-driven one used, would result in very little difference in mean motions. This arises because the low-strain damping in the deep profile (between 200 and 3000 feet deep) largely controls the response for frequencies above approximately 1 Hz. This is constrained by the use of a kappa value based on observations of motions recorded in the embayment. The applicant considered the empirical kappa value it employed to be conservative since a deeper sedimentary column (over 3 kilometers at the site) over hard rock is expected to show larger kappa values (more cumulative damping) in comparison with shallower sedimentary columns.

Figure 2.5.4-13 shows the base case shear wave velocity profile to the analysis depth of 1 kilometer along with two extreme cases for extrapolation to the hard rock shear wave velocity of 2.8 km/s. The slow profile assumes a uniform extension of the velocity just beneath the site to 1 kilometer, with a jump to the hard rock value of 2.8 km/s. As another extreme alternative, the fast profile assumes a linear (modeled as a stair step) gradient with depth, reaching 2.8 km/s at a depth of 1 kilometer. Based on general experience with measured shear wave velocity profiles in large sedimentary basins, the applicant expected the slow and fast profiles to reflect extreme (i.e., low likelihood) conditions, particularly with regard to the local and regional overall geology.

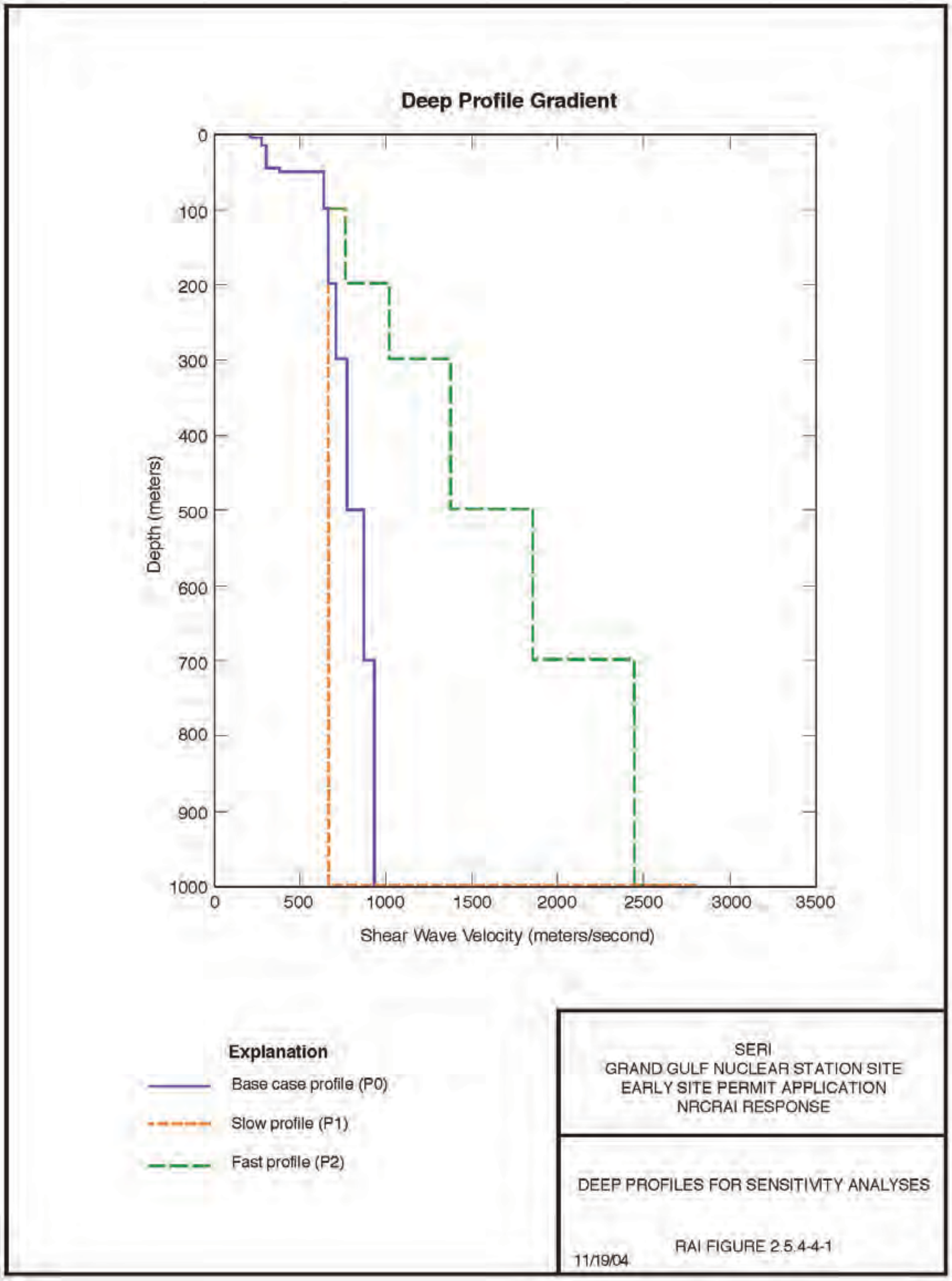


Figure 2.5.4-13 (Figure 2.5.4-4-1 of the RAI response)
Deep Profiles for Sensitivity Analyses

To compare the median motions computed for the three profiles, the applicant stated that it used the 1- to 2-Hz-scaled 10^{-5} APE spectrum as the control motions to excite the column at low as well as high frequency since it is quite similar to the UHRS and sufficiently broad band (i.e., large magnitude) compared to the UHRS. Figures 2.5.4-14 and 2.5.4-15 show the resulting soil median motions in both the logarithmic and linear spectral axes, respectively. As expected, little difference exists between the motions computed for the three profiles from 0.3 to 100 Hz. At peak acceleration, the fast and slow profiles show about a 5-percent increase and decrease, respectively, in motions, as compared to the base case profile. The motions reflecting the fast profile generally exceed those of the base case by about 5 percent. The largest exceedance is near a resonance at about 3 Hz, at which the fast profile motions exceed the base case motions by about 10 percent. Based on regional geology as well as experience with similar geology in deep basins, this fast profile is extremely unlikely and would receive very low weight in a hazard analysis. The applicant stated that these site response analyses illustrate the general insensitivity of the motions to the nature of velocity gradients likely to exist in the deep materials beneath the site.

In SSAR Section 2.5.4.1.3, the applicant summarized velocity properties for the various layers of the shallow soil profile. Section 7.7.1 of ER-02 indicates that the loess has a shear wave velocity that ranges from 243.84 m/s (800 fps) to 274.32 m/s (900 fps). The borehole logger data in Appendix D to ER-02 show values as low as 182.88 m/s (600 fps). Table ER-02-4 indicates values that range from about 182.88 m/s (600 fps) to over 426.72 m/s (1400 fps). Section 8.2 of ER-02 indicates that the BE is 234.7 m/s (770 fps). In RAI 2.5.4-8, the staff asked the applicant to explain why the mean velocity values for all the material layers are not approximately centered on the ranges listed in ER-02 Table 8.2. In its response, the applicant stated that the staff noted apparent inconsistencies in the range of shear wave values presented for the various site stratigraphic units (loess, Upland Complex Alluvium, Upland Complex Old Alluvium, Catahoula formation) in the SSAR and ER-02 text, tables, and figures. These apparent inconsistencies are related to (1) rounding of values for general descriptions of unit properties in the text, (2) presentation of localized extreme values on tables and figures (before data smoothing), and (3) the use of different depth intervals over which averaged values are presented that sometimes cross stratigraphic boundaries.

The staff also asked the applicant to compare the values of shear wave velocity to the BE, UB, and LB values used in the site response calculations. In its response, the applicant stated that Figure 2.5.2-4, which replicates SSAR Figure 2.5-60, presents the BE velocity profile adopted for the site soil response analysis. This velocity profile is based on a visual average of the composite P-S velocity profiles from the three ESP boreholes. It is an averaged, smoothed profile that does not use extreme values. The BE profile consists of five separate interval velocities that incorporate velocity data that were binned differently than the various general and estimated velocity ranges described for different stratigraphic units. The BE interval velocities of the site soil response velocity profile are not set at stratigraphic unit boundaries but rather are assigned at visually determined velocity breaks in the composite P-S profile. For this reason, the BE site soil response average velocities are not centered on the mean values listed for soil layers in ER-02 Table 8.2.

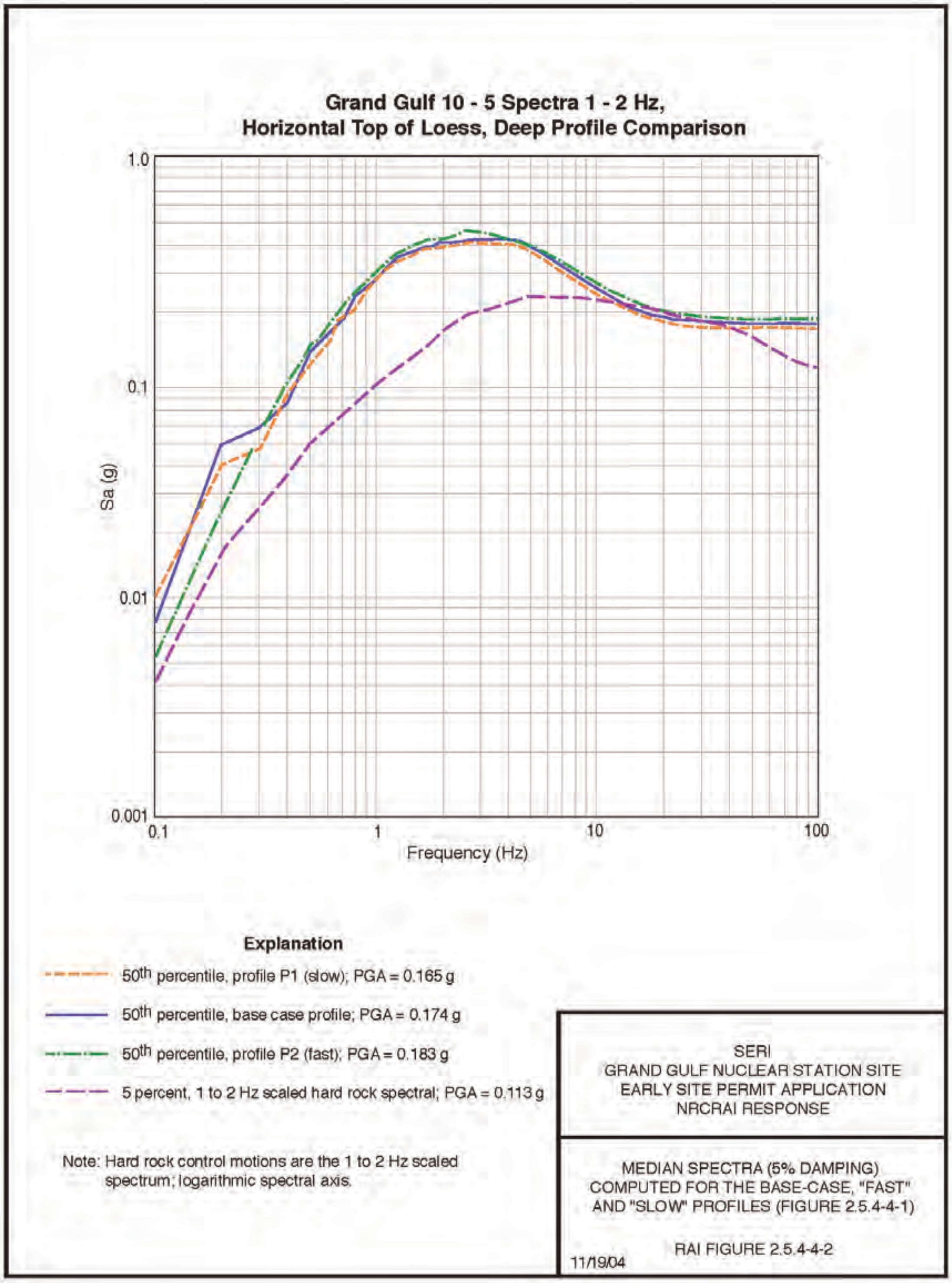


Figure 2.5.4-14 (Figure 2.5.4-4-2 of the RAI response)
Median Spectra (5% Damping)
Computed for the Base-Case, "FAST"
and "SLOW" Profiles (Figure 2.5.4-4-1)

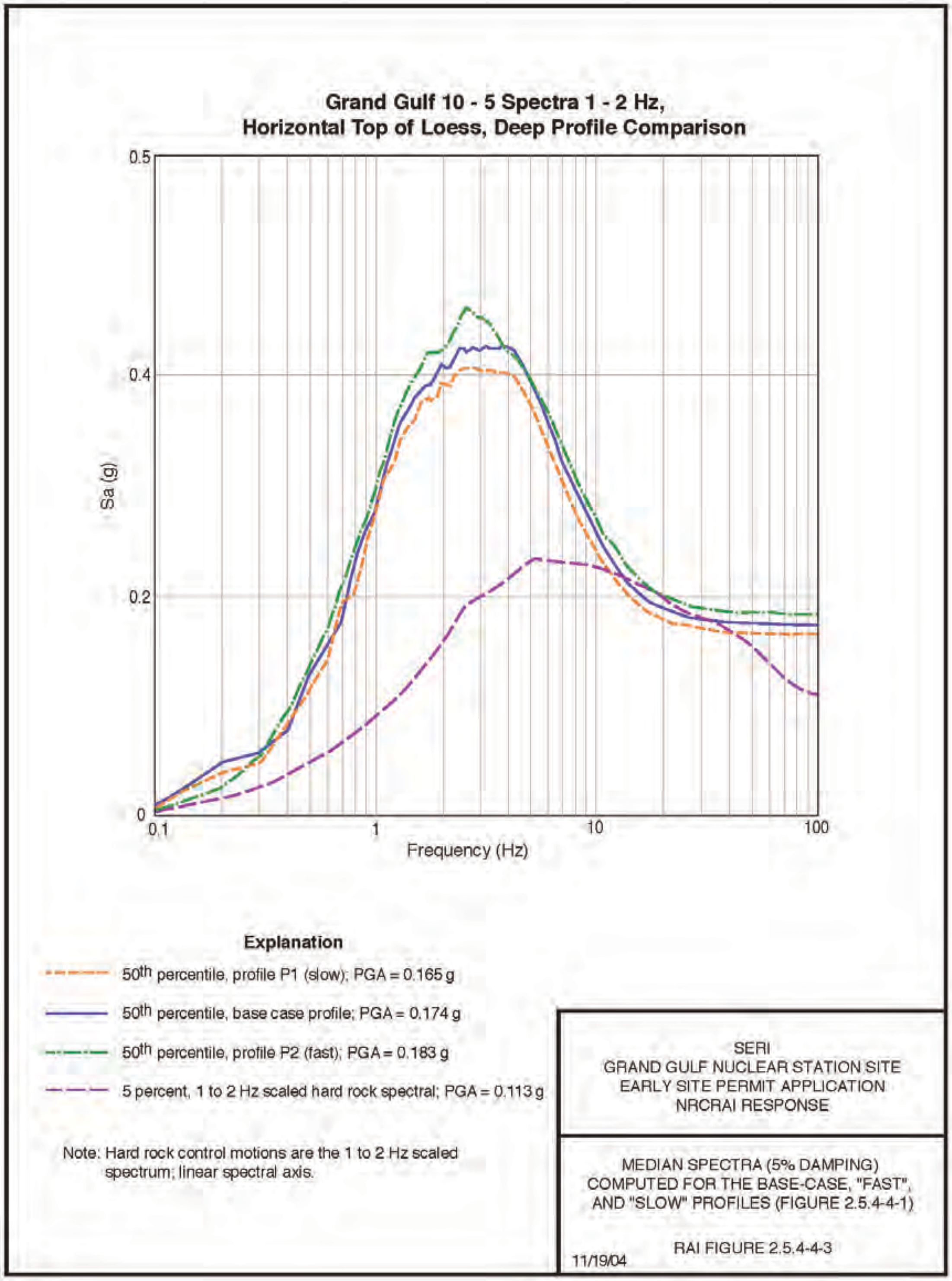


Figure 2.5.4-15 (Figure 2.5.4-4-3 of the RAI response)
Median Spectra (5% Damping)
Computed for the Base-Case, "FAST",
and "SLOW" Profiles (Figure 2.5.4-4-1)

In ER-02 Section 3.3, the applicant stated that it planned to place all safety-related facilities on the Upland Alluvium or the Old Alluvium with an average shear wave velocity of at least 304.8 m/s (1000 fps). The suspension logging data provided in Appendix D to ER-02 indicate that the measured shear wave velocities are as low as 152.4 m/s (500 fps) at depths of up to120 feet (e.g., see results for Boring B-1). This depth is well below the planned depth of foundations indicated in the SSAR. As discussed in RAI 2.5.4-11, because measured shear wave velocities are based on the results from only three borings, and considering the normal variability anticipated in shear wave velocity, the staff asked the applicant to evaluate the impact of such a velocity cutoff on the minimum depth for future siting, particularly because some of the advanced reactor designs may require a minimum shear velocity of 304.8 m/s (1000 fps). In its response, the applicant stated that individual measurements of shear wave velocity obtained using the P-S suspension logger are as low as 161.54 m/s (530 fps). However, these are localized zones in a single boring and generally occur within the loess or the Upland Complex Alluvium, although they also occur less commonly in the Old Alluvium. The lower velocity zones are typically less than 10 to 20 feet thick, representing a relatively small percentage of the strata within each stratigraphic unit, and zones exhibiting greater wave velocities bound the lower velocity zones. The deepest occurrence of a lower velocity zone is at a depth of about 154 feet, and the minimum elevation at the base of a lower velocity zone is at an elevation of about !7.4 feet. The lower velocity zones have zone-specific average wave velocities in the range of 231.65 m/s (760 fps) and 295.66 m/s (970 fps).

The applicant further noted that the minimum required shear wave velocity at the foundation level for all example reactor types considered is 304.8 m/s (1000 fps). Data from the existing three deep borings at the ESP site suggest that this criterion is marginally met in the Upland Complex Alluvium and is easily met in the Upland Complex Old Alluvium. In accordance with RG 1.132, the applicant should drill confirmatory borings at the site during the COL phase site investigations. These borings should include additional borehole velocity surveys to verify the velocity profile under the foundation footprint and to confirm the depth of the target foundation-bearing strata that satisfy the selected plant shear wave velocity design criteria. Based on the existing UFSAR and ESP data, the applicant believed that the depth range to deposits exhibiting a shear wave velocity of 304.8 m/s (1000 fps) will not vary significantly from the results presented in the SSAR.

In ER-02 Section 7.5 and related SSAR sections, the applicant stated that the difference in blow counts for the SPT between previous field programs and the current program performed for the ESP site may result from the difference in hammer equipment used for the samples. As stated in RAI 2.5.4-12, because only three borings were taken for the ESP program, conclusions for this evaluation rely heavily on the previous site investigations. The staff requested that the applicant quantify and evaluate the impact of this difference (such as taking a new boring adjacent to an old boring using new equipment).

In its response, the applicant stated that it performed 38 SPT tests in the ESP borings and the GGNS licensee performed 1069 SPT tests on the UFSAR borings. The applicant segregated the SPT data into the various ESP stratigraphic units from which it obtained the samples. A comparison of the tabulated data demonstrates that the UB and LB ranges of the UFSAR blow counts are about 110 to 250 percent higher than those in the ESP borings for loess, 150 to 300 percent higher for the Upland Complex Alluvium, and 110 to 120 percent higher for the Upland Complex Old Alluvium.

The applicant obtained SPT blow counts for the ESP boreholes using an automatic trip hammer, which is a very consistent and highly efficient delivery system. The GGNS licensee did not document the delivery system for SPT sampling in the UFSAR boreholes. However, based on the vintage of the exploration program (1971–1972), it is unlikely that the licensee performed the UFSAR sampling with automatic trip hammer equipment that did not come into standard usage until the mid-1980s. Rather, it probably obtained the samples using a less efficient and consistent cathead or wireline delivery system. Based on standardized hammer efficiency correction factors, the automatic hammer system used in the ESP investigations has an estimated efficiency factor of approximately 90 percent, compared to the lower efficiency of about 60 percent for the UFSAR SPT testing with an assumed cathead or wireline delivery system. The applicant noted that the efficiency difference in SPT hammer sampling delivery systems could largely account for the markedly lower SPT blow count ranges obtained in the ESP borings for the same stratigraphic units than those sampled in the UFSAR borings.

The applicant further stated that it used the SPT test data for the ESP in a qualitative and relative sense to perform an initial characterization and evaluation of the site subsurface conditions rather than for engineering design or quantitative geotechnical analysis. In this context, the ESP SPT data would provide a conservative general assessment of the material properties. In accordance with RG 1.132, COL-phase investigations will include additional borings and SPT testing throughout the selected power plant footprint area to increase the SPT database. The applicant expected that selected COL borings will be performed with energy calibration equipment to allow accurate measurement and adjustment of the hammer blows for design-level geotechnical analysis and development of foundation and excavation design parameters. The COL borings should also include index borings at the ESP boring locations to allow calibration among these boring sets. The applicant marked the ESP borings with steel stakes set in cement grout at the borehole locations and also performed a survey for future reference purposes.

In ER-02 and SSAR Section 2.5.4.1.6, the applicant stated that it shipped the samples to its contractors either by an automobile or by FedEx. The description in Appendix G to ER-02 indicates that all samples examined for dynamic testing had the appearance of competent and intact materials. Section 11.0 of ER-02 indicates that samples were carefully extracted from the borehole and presumably carefully shipped to the contractor. However, the report notes that the results of the dynamic testing indicate some effect of sample disturbance. In RAI 2.5.4-13, the staff asked the applicant to explain what measures it took, if any (aside from these qualitative statements), to ascertain whether any significant disturbance occurred during the sampling, transportation, or laboratory extrusion process. Since the static testing program includes consolidated undrained (CU) triaxial tests, the staff asked the applicant to address the concern on sample disturbance. In its response, the applicant explained that it carefully extracted sample sets (pressed Shelby tube samples and driven modified California samples) from the borings. Once extracted, the samples were inspected, capped with plastic lids and electrical tape, and sealed with paraffin according to the project protocol. The applicant placed the Shelby tube samples in foam-padded wood crates and delivered them to its contractor using a passenger van. The applicant also placed the modified California tube samples in padded cardboard boxes and shipped these to the testing laboratory.

Based on its review of the laboratory reports and test results, the applicant believed that the stress relief during borehole extraction, movement of shells or gravel-size clasts in the samples during sampling, or movements (e.g., tilting) during laboratory testing might have caused some disturbance to the samples. These effects do not appear to be the result of handling or transportation of the samples because the documentation of the transportation and initial laboratory inspection upon receipt does not indicate any disturbance of the samples.

The applicant noted that the ratios between the shear wave velocities measured in the laboratory and those measured in the field at the same depth for the test samples indicate possible extraction-related sample disturbance. The applicant also suggested that it accounted for these effects in its interpretation of the appropriate shear modulus reduction and damping curves. The applicant did not make corrections to the static triaxial test results since it used these only as index tests to generally compare relative properties between different stratigraphic units and against the triaxial tests performed for the UFSAR. Furthermore, the applicant did not use the triaxial test results as input parameters to calculate foundation or design criteria and stated that the COL applicant would develop these parameters after the selection of a specific reactor, in accordance with RG 1.138.

The applicant presented, in Appendix D to ER-02, the results of the P-S borehole logging in the three boreholes. From these data, the applicant generated P- and S-wave profiles to depths of 180 to 225 feet. In RAI 2.5.4-14, the staff identified the following two issues:

(a) The applicant stated on page 7 of Appendix D that the "shear wave data was of excellent quality in the three boreholes." The staff asked the applicant to provide the basis for this judgment. In addition, the staff requested that the applicant provide a corresponding evaluation for the quality of the compressive wave (P-wave) data.

(b) The plots of P-wave velocity with depth demonstrate the relatively rapid increase on the P-wave that one would normally expect to occur near the ground water table. The results from Borings 1 and 3 show this characteristic velocity increase. However, the results from Boring B-2A show the characteristic rise followed by a significant reduction at a depth of approximately 70 feet. The staff asked the applicant to explain the cause of this anomaly.

In its response to part (a) of the RAI, the applicant noted that GeoVision (the applicant's contractor) maintained a quality assurance/quality control program that includes a comparison between the independently acquired source-to-receiver velocities and receiver-to-receiver velocities, as well as an independent review by a second geophysicist. GeoVision's report discusses the good agreement between the source-to-receiver and receiver-to-receiver interval velocities for both shear wave and P-wave velocities in each borehole, providing verification of the general good data quality. Therefore, the statement as to good data quality refers to both the shear wave and P-wave data, and no implication is made regarding a lower quality with respect to the P-wave data. The applicant also stated that it independently reviewed the GeoVision P-S survey data and found the correlation between source-to-receiver and receiver-to-receiver data to be very good in all boreholes. Additionally, the applicant's geologists observed the GeoVision P-S survey field setup, calibration, and logging for each borehole. Based on the field review, the applicant concluded that its contractor performed the surveys carefully and according to the approved ESP Project Instruction PI-05.

In its response to part (b) of the RAI, the applicant stated that the initial rise in the P-wave velocities in Boring William Lettis and Associates (WLA) B-2A occurs near the contact between the loess and the Upland Complex Alluvium; the applicant believed that this initial rise results from a perched water table within the base of the loess above the Upland Alluvium. The permanent ground water table is delineated by a P-wave rise within the Upland Alluvium approximately 15 feet deeper in the boring, which does not drop back down, and correlates well with the P-wave rise that marks the top of the ground water table in Borings WLA B-1 and B-2.

In Appendix F to ER-02, the applicant presented the results of standard geotechnical identification tests and CU triaxial strength tests. In RAI 2.5.4-15, the staff identified the following observations:

(a) The sample descriptors provided with the grain size distribution curves present descriptions such as “FINE SAND w/Silt” (see, for example, B1-S22 as well as others) for cases in which the sample contains less than 10 percent fines. The staff asked the applicant to demonstrate that such descriptors will not mislead evaluation based on verbal descriptions.

(b) The tables also include USCS descriptors (such as ML or CL) for samples for which no Atterberg Limits were determined. For these cases, the staff asked the applicant to provide the basis for developing such classifications.

(c) For some samples, the applicant performed only one CU test from which estimates of strength parameters (cohesion and friction angle) are listed. The staff asked the applicant to provide the basis for such judgments.

In its response to this RAI, the applicant explained that it based these soil classification issues on initial field visual-manual descriptions following the guidelines of American Society for Testing and Standards (ASTM) D2488, “Practice for Description and Identification of Soils (Visual-Manual Procedure).” The applicant modified these field descriptions, as necessary, for the final soil classifications based on the results from laboratory index testing, following the guidelines of ASTM D2487, “Test Method for Classification of Soils for Engineering Purposes.” The applicant also stated that it configured the laboratory testing program to confirm or refine the field descriptions and targeted it toward field samples that had uncertain descriptions or soils that fell on the border between two different possible classifications.

The final soil classifications presented in SSAR Table 2.5-7 indicate the final evaluations made by field geologists to accommodate differences between field and laboratory classifications. The applicant, in some cases, kept or slightly modified the field classification, rather than adopting the laboratory classification. The applicant also decided to keep field classifications based on the perceived quality and quantity of the laboratory data, as well as the level of confidence in the field description.

The applicant continued to state that it limited the number of samples submitted for laboratory testing by the number and quality of samples that could be extracted from the borings, as well as on the basis of budget and schedule. Therefore, the applicant extrapolated the results from the laboratory tests by indexing tested samples to the field descriptions, and then developing correlations for other samples that were not tested in the laboratory.

In response to RAI 2.5.4-15a, the applicant stated that it classified the referenced sample, B-1-22, as a fine sand with silt (SP-SM); the sample contains 9.5-percent silt and 2.2-percent clay. The applicant believed that the USCS classification for this sample, “with silt,” is correct. The USCS specifies the use of this descriptor when a sand contains between 5 and 12 percent silt. The applicant adhered to the USCS descriptor because it is a universally applied standard soil classification scheme for geotechnical and engineering applications and therefore should not be misleading. To respond to this RAI, the applicant performed another round of review for all the ESP soil classifications and found that the classification for some sand samples was incorrect with respect to the grading (e.g., poorly graded versus well graded (SP/SW-SM)). In addition, some fine-grained soil classifications require a change from lean clay to silt (CL/ML) or vice versa. The applicant presented the modified classifications in SSAR Table 2.5-7 and explained that these soil classification modifications do not change the findings of the investigation or require changes to the analysis input because the overall descriptions for the general properties of the major stratigraphic units remain unchanged. Most of the changes pertain to soils that were on the border of two classifications. In addition, the applicant presented the actual soil testing results in SSAR Table 2.5-7 and included these results in Appendix F to ER-02 for evaluation purposes.

In its response to RAI 2.5.4-15b, the applicant explained that, based on the soil behavior observed by field visual-manual description methods or extrapolated from similar samples that were subjected to Atterberg Limits testing, it made soil classifications for clay or silt samples that did not have Atterberg Limits. The applicant performed 18 Atterberg Limits tests on selected fine-grained ESP samples; these tests included multiple tests from each of the site stratigraphic units (loess, Upland Complex Alluvium, and Old Alluvium) and one sample from the Catahoula formation claystone. With the exception of the Catahoula formation sample that is classified as a fat clay (CH), all other samples fall within the classification zones for lean clay or silt (CL or ML). These samples also exhibit similar visual-manual properties. On this basis, the applicant believed that it is reasonable to extend the classification of lean clay and silt to the samples that have similar field behavior.

In its response to RAI 2.5.4-15c, the applicant also noted that, among the 18 samples prepared for CU triaxial shear testing, only 10 were found to be suitable for triaxial testing or remained intact when placed in the testing load cell. Only one 3-point CU test series and two 2-point CU test series were possible on stratigraphically adjacent intact samples. The applicant obtained the remaining three samples from nonadjacent stratigraphic units and evaluated them using single-point CU tests.

The applicant plotted the triaxial test results on p-q (stress path plot where $p = (s_1\text{-}s_2)/2$ and $q = (s_1+s_2)/2$) and Mohr circle (t-s) plots to estimate the cohesion and friction angle components for the shear strength failure envelopes. For tests with a single point, the applicant defined the failure envelope by a line extending from the graphical origin (zero cohesion) and constrained it by a single tangential contact point on the Mohr circle. For multipoint tests, the applicant used a best fit between tangential points on the multiple Mohr circles to estimate these parameters.

The applicant further noted that the single-point test results plot consistently when transferred to the multipoint test plots. Additionally, the one-point and multipoint stress-strain curves for the loess samples have generally similar shapes as the single and multipoint stress-strain curves. The applicant considered these strength values to be reasonable based on its experience

performing multipoint triaxial tests on similar materials. These values also fit within the range for soils with similar classifications reported in the literature (e.g., Kulwahy, F.H. and Mayne, P.W., EPRI Report No. EL 6800, "Manual on Estimating Soil Properties for Foundation Design," 1990). As described in its response to RAI 2.5.4-13, the applicant used the ESP triaxial test results only as index tests to generally compare relative properties between different stratigraphic units and also to compare against triaxial tests performed for the UFSAR. The applicant did not use triaxial test results as input parameters to calculate foundation or design criteria; therefore, it considered extrapolation of the single-point test results to estimate shear strength parameters to be valid for the ESP study.

In Appendix G to ER-02, the applicant presented the results of dynamic laboratory testing on six intact samples obtained from Borings B-1, B-2, and B-3 taken at the ESP site. These tests evaluated the linear and nonlinear shear modulus and material damping characteristics of the samples. The dynamic laboratory tests included both RC and TS conducted at different confining pressures and maximum strain levels. The samples tested were all fine-grained soil samples, four having low plasticity (less than 5 percent) and two having more plasticity (PI = 12–13 percent). Table 3 of Appendix G (as well as Table ER-02-6) indicates that three of the samples were confined at pressures based on an assumed value of K_o of 0.5 and three were tested at an assumed value of 1.0.

In RAI 2.5.4-16, the staff requested the applicant to provide the basis of these selections and compare the resulting pressures with current estimated in situ stress levels. In its response, the applicant explained that it performed dynamic laboratory testing for six samples from Borings WLA B-1, B-2, and B-3. In all six cases, the applicant consolidated the test specimens to an isotropic stress equal to the mean estimated in situ stress based on an estimate of k_0, the coefficient of lateral earth pressure. The applicant also consolidated the two loess specimens assuming a k_0 value of 0.5 and the other four test specimens (two each for the Upland Complex Alluvium and the Old Alluvium) assuming a k_0 value of 1.0. The applicant noted that the RAI incorrectly states that three samples were assigned a k_0 value of 0.5 and three samples used a k_0 value of 1.0. The applicant selected a k_0 value of 0.5 for the loess because this material is younger (less consolidated) than the Upland Complex Alluvium and the Old Alluvium. As discussed in the response to RAI 2.5.4-6, in retrospect, it would obtain a better match between the shear wave velocities of the loess measured from the field and from the laboratory using the higher k_0 value of 1.0 for the laboratory samples. This conclusion is reflected by the improved agreement between the field and laboratory velocities for the loess samples that were tested at a confining pressure of 4 times the estimated in situ stress, as shown in SSAR Table 2.5-26. The k_0 value of 0.5 for the loess, therefore, does not sufficiently account for the significantly overconsolidated nature of these deposits. As discussed in the response to RAI 2.5.4-6, the applicant gave a greater weight to the results from the loess samples tested at 4 times the estimated in situ confining stress during the evaluation and selection of the appropriate EPRI 1993 shear modulus reduction and hysteretic damping curves for the site response analysis.

2.5.4.1.2 Site Ground Water Occurrence

SSAR Section 2.5.4.2 indicates that information on the ground water table in the ESP site area is only available from the three borings taken during the ESP site investigation program. However, since water was injected into these borings during the drilling process, direct

measurement of ground water tables was not possible. The ground water table can be approximately inferred from the P-S seismic velocity logging conducted in these three borings during the geophysical testing program described previously. These indicate that the ground water table is at a depth of approximately 70–100 feet below grade where the P-wave velocity data show a significant increase in value in all three borings. This increase in velocity is typically associated with saturation of the soil and indicates the effect of ground water on soil compressibility. The SSAR also indicates that the ground water table exhibits a relatively shallow drop of about 1 foot per 100 feet, indicating a gradual flow of the ground water to the southwest. The applicant did not describe the source of this information nor discuss issues for the control of ground water and the influence of ground water on construction settlements in the ESP site area. The applicant implied that this information will be of interest during the COL stage of the project.

2.5.4.1.3 Response of Soil to Dynamic Loading

Based on the results of the geotechnical site investigation, the applicant stated that the geologic materials underlying the proposed site location are not prone to dynamically induced failure or excessive strength loss or deformation. The applicant, by referencing SSAR Section 2.5.4.4, concluded that the susceptibility of soil deposits to liquefaction is low. The applicant also referred to SSAR Section 2.5.4.5.2 for the preliminary assessment of bearing capacity and settlement and SSAR Section 2.5.4.1.4 for the dynamic shear modulus reduction and damping of site soils. In SSAR Section 2.5.4.3, the applicant stated that the site is stable and will not be prone to dynamically induced failures. The applicant further noted, in Section 3.3 of ER-02, that the site does not show any indications of dissolution cavities or sinkholes. Nevertheless, SSAR Section 2.5.1.2.3.1.2.3 indicates that calcareous clays, limestone, and marl formations that are prone to dissolution cavities or sinkholes exist beneath the surface at the ESP site and may even be exposed in the site vicinity. None of the borings shown in the profiles (old or new) reaches to these depths. In RAI 2.5.4-9, the staff asked the applicant to explain its statement that the site is not susceptible to such potential long-term problems. In its response, the applicant explained that calcareous clay, limestone, and marl from the Glendon Limestone and Bryam Marl only exist at depths greater than 390 feet within the Vicksburg Group. These represent the shallowest significant occurrence of calcareous rocks beneath the ESP site, and the Vicksburg Group calcareous units do not appear to be susceptible to karstic dissolution in the site area. Evaluation of aerial photos, topographic maps, and field reconnaissance shows that there are no karstic features either in outcrop areas of this group or expressed within the overlying unconsolidated deposits. In addition, foundations for the existing GGNS unit have performed satisfactorily and have not experienced any settlements related to potential dissolution of strata within the Vicksburg Group at depth.

The applicant's response also involves three separate evaluations, namely (1) evaluation and documentation of the presence or absence of karstic features in the site area (5-mile radius), (2) evaluation and documentation of the presence or absence of karstic features in outcrops of the Vicksburg Group in the site vicinity (25-mile radius) and site region (200-mile radius), and (3) evaluation of the zone of influence of any new proposed foundation on the Vicksburg Group strata.

To evaluate the presence or absence of karstic features in the ESP site area, the applicant reviewed existing literature; examined topographic maps, aerial photos, and orthophotographs; and performed field reconnaissance. The results from these studies show that features indicative of karstic processes are not present in the ESP site area. None of these features have developed within the Pliocene to Pleistocene unconsolidated surficial deposits in the ESP site area. The geologic map shows that no limestone, marl, or calcareous units are exposed in the ESP site area and that the Vicksburg Group does not outcrop in the ESP site area. The applicant prepared geologic maps compiled from existing literature, interpretation aerial photos, and field reconnaissance. The applicant concluded that karst-related geomorphic or topographic features are not present in the ESP site area. The applicant further noted that, although the absence of karstic features at the ground surface does not preclude the potential for dissolution at depth, the absence of such features in the overlying unconsolidated sediments indicates that possible deep dissolution has not extended upward into the thick cover sediments. Based on its evaluation of existing information, the applicant concluded that karstic processes have not influenced the ground surface or shallow subsurface in the ESP site area and that these processes are not likely to affect the ESP site area in the future.

To evaluate the presence or absence of karstic features in outcrop exposures of the Vicksburg Group in the site vicinity and site region, the applicant noted that limestones of the Vicksburg Group crop out in a narrow belt that extends in an east-west direction across the central part of the state of Mississippi. Within the Vicksburg area, the nearest units of the Vicksburg Group crop out about 20 miles north of the ESP site. Available geologic and topographic maps of this area do not indicate karst-related topographic or geomorphic features (e.g., sinkholes, circular depressions) within the outcropping areas of Vicksburg Group carbonate rocks. The maps do define several concentric lakes within the Vicksburg Group outcrop areas that are up to approximately 100 meters in diameter. However, these lakes are also developed in areas where the noncalcareous Catahoula formation crops out (see RG DG-1109, "Laboratory investigations of soils and rocks for engineering analyses and design of nuclear power plants," issued August 2001; NUREG/CR-6728; and NUREG-1488, "Revised Livermore Seismic Hazard Estimates for Sixty-Nine Nuclear Power Plant sites East of the Rocky Mountains," issued 1994), suggesting that the lakes are not associated with carbonate dissolution processes. No records show any of the lakes to be associated with karst formation (see Draft RG DG-1101, "Site Investigations for Foundations of Nuclear Power Plants," issued February 2001).

The applicant stated that, although records do not indicate karst development in the Vicksburg Group outcrops in the vicinity of Vicksburg and the Grand Gulf site, karst features are referenced as occurring within the Vicksburg Group in the area east of Jackson, Mississippi, approximately 50 miles northeast of the Grand Gulf ESP site. Although records do not indicate caves or evidence of karst development, USGS classified the Vicksburg Group as having a potential for karstic development in a regional map compilation of the Appalachian region. The USGS classification is based on descriptions of regional lithology that include descriptions of calcareous units within the Vicksburg Group and do not include field or other verification of karst development in these units. Instead, USGS based its statement regarding the potential for karst development on carbonate lithologies of regional units with a conservative bias towards inclusion of any potentially soluble units.

The applicant further noted that the zone of influence of any new proposed foundation on the Vicksburg Group strata will be well away from any potential karst formations. The ESP plant foundation would have a minimum separation distance of about 300 feet from the top of the limestone. The applicant expected that the foundation influence zone extends about 100–200 feet below the base of the foundation. In accordance with RG 1.132, the applicant will conduct additional site investigations during the COL phase of the project. These investigations should include deep borings in the planned foundation footprint and geotechnical analysis of the bearing pressure and load distribution in the foundation soils based on the specific geometry, embedment, and loading of the selected reactor. In addition, the applicant stated that in case any calcareous clays, limestone, or marl deposits are found within the incremental pressure bulb created by the planned construction, the applicant should drill further borings at sufficiently close spacing to confirm the absence of dissolution cavities within the foundation-bearing zone.

2.5.4.1.4 Liquefaction Potential and Seismic Site Stability

Section 2.5.4.4 of the SSAR indicates that the deposits underlying the proposed site location range in age from Miocene (Catahoula formation) to Pleistocene (loess silts). As mentioned above, these deposits all appear to be overconsolidated. The applicant did not identify any Holocene materials or relatively loose sands or silts that may be susceptible to liquefaction at the ESP site location. The applicant also discussed the paleoliquefaction indicators in Section 2.5.2 of the SSAR, and these indicators are encountered in relatively young Holocene floodplain deposits. The applicant stated that no such deposits underlie the proposed site, and no paleoliquefaction features have been reported in the ESP site vicinity. Soils below the ground water table that are planned to provide foundation support are relatively dense, overconsolidated, and relatively old and thus are not susceptible to liquefaction.

2.5.4.1.5 Static Site Stability

Section 2.5.4.5 of the SSAR discussed several issues associated with the foundation conditions that must be addressed for design and construction of new facilities at the ESP site.

<u>Bearing Capacity and Settlement</u>. SSAR Section 2.5.4.5 indicates that the COL applicant will evaluate issues associated with allowable bearing capacity, excavation rebound, construction settlements, and long-term foundation settlement during the COL phase of the project. This SSAR section also indicates that the applicant would locate the foundations of Category I facilities in the Upland Alluvium, although other sections indicate that the Catahoula is the primary bearing strata of interest. Based on the known site conditions, the applicant noted that bearing capacity and settlement properties of the foundation soils are not expected to be a concern for a new nuclear power plant and should not provide any obstacles to construction.

The applicant has provided no specific calculations to support this conclusion. SSAR Section 2.5.4.5 indicates that new facilities will be found at a depth of about 50 feet below grade in the Upland Alluvium. However, the SSAR also states that the soils below the foundations should have an average shear wave velocity exceeding 304.8 m/s (1000 fps). If soft or compressible soils exist below the foundations, the SSAR indicates that the COL applicant would excavate to the required depth and replace these materials with appropriate engineered fill having the required characteristics. The applicant did not mention in the SSAR anticipated

levels of contact pressures for the prospective facilities from dead, live, and seismic loading conditions and how these may compare with approximate allowable pressures for the site soils.

This SSAR section also indicates that excavation will occur through the loess and possibly extend into some of the Upland Alluvium deposits. This excavation will result in the removal of at least about 7 kips per square foot of overburden. The applicant expected several inches of heave or rebound to occur as a result of this excavation. The applicant further stated that this heave should not be sufficient to threaten stability of the excavation. Such excavation heave was encountered during construction of GGNS, and the UFSAR indicates that several inches of heave and recompression occurred during this process.

<u>Lateral Earth Pressures and Hydrostatic Loading</u>. SSAR Section 2.5.4.5.2 indicates that the COL applicant would evaluate issues associated with lateral earth and water pressures during the COL phase of the project. The applicant also noted that several inches of elastic rebound may be associated with the relatively deep site excavations that are planned and that this rebound would be expected to be reversible, presuming that the new structures are fully compensated designs. Assuming that the current GGNS powerblock structures are far enough from the ESP site, the staff, in RAI 2.5.4-10, asked the applicant to demonstrate that no other facilities (piping, conduit, etc.) exist in the ESP area that may be affected by such surface movements. In its response, the applicant explained that it anticipated that a new facility at the ESP site would maintain the existing plant grade of 132–134 feet of elevation. In addition, new facilities would require embedment below the loess deposits into the Upland Complex Alluvium or the Old Alluvium to an elevation where the average shear wave velocity is at least 304.8 m/s (1000 fps). This likely would require excavations to depths of approximately 60–100 feet or deeper. The in situ confining stresses at these embedment depths range between about 24,412 kg/m^2 (5 ksf) to 48,824 kg/m^2 (10 ksf). The confining stress at the maximum manufacturer-listed depth of 140 feet is on the order of approximately 58,589–73,236 kg/m^2 (12–15 ksf). These estimated in situ pressures are in the range of, but somewhat lower than, the minimum static bearing pressure requirements for the various reactors, suggesting that the foundations will be partly compensated.

The applicant also stated that up to several inches of predominantly elastic rebound may occur as a result of excavations for reactor foundations. The applicant estimated this value based on a review of the performance of the existing GGNS containment and turbine buildings. Comparison of the measured rebound and settlement data for GGNS shows that the settlement recovered a significant percentage of the rebound. The applicant further stated that no existing safety-related Class I facilities fall within, or adjacent to, the estimated ESP influence zone. Based on this evaluation, construction and foundation loads from a new plant at the ESP site would not impact the existing power plant structures, operation, or safety. Tied-back sheet piles or other bracing systems would likely support deep excavations in a manner similar to the excavations for the existing power plant.

In accordance with RG 1.132, the applicant will perform additional site exploration, laboratory testing, and geotechnical analyses during the COL phase of the project after the selection of a plant design to verify site conditions for foundation design geotechnical analyses and determination of embedment criteria. These borings and additional engineering analyses should include evaluation of deformation moduli for the subsurface materials to refine estimates of the influence zone related to the selected reactor and foundation configurations. The COL

investigations should also consider development of an instrumentation and monitoring program to measure actual construction-related soil movements.

2.5.4.1.6 Design Criteria

Section 2.5.4.6 of the SSAR indicates that the COL applicant would develop the geotechnical criteria needed for the design of new plant foundations, temporary construction, and excavation support during the COL stage. The geotechnical material properties currently available from the ESP study would be supplemented with additional information developed during the COL stage. The SSAR does not discuss geotechnical design criteria (e.g., allowable bearing capacity, safety factors) or criteria that pertain to structural design (e.g., wall rotation, sliding, and overturning).

2.5.4.2 Regulatory Evaluation

SSAR Section 2.5.4 describes the applicant's evaluation of the stability of the subsurface materials and foundations at the ESP site. The applicant stated that it developed the geological, geophysical, and geotechnical information used to evaluate the stability of the subsurface materials in accordance with the guidance presented in 10 CFR Part 52 and Appendix S to 10 CFR Part 50, as well as the requirements in 10 CFR 100.23. In addition, the applicant stated that the ESP program conducted for this site partially conforms to the guidance provided by RGs 1.132 and 1.138.

The staff considered the regulatory requirements in 10 CFR 100.23(c) and 10 CFR 100.23(d)(4) in its review of SSAR Section 2.5.4. Pursuant to 10 CFR 100.23(c), the engineering characteristics of a site and its environs must be investigated in sufficient scope and detail to permit an adequate evaluation of the proposed site. According to 10 CFR 100.23(d)(4), siting factors such as soil and rock stability, liquefaction potential, and natural and artificial slope stability must be evaluated. Section 2.5.4 of RS-002 provides specific guidance concerning the evaluation of information characterizing the stability of subsurface materials, including the need for geotechnical field and laboratory tests as well as the geophysical investigations.

2.5.4.3 Technical Evaluation

This section provides the staff's evaluation of the geotechnical investigations conducted by the applicant to determine the static and dynamic engineering properties of the materials that underlie the Grand Gulf ESP site.

Using Attachment 2 (Sections 2.5.4 and 2.5.5) to RS-002 as guidelines, the staff performed its review of SSAR Section 2.5.4 and ER-02, referenced in the SSAR. In order to thoroughly evaluate the surface faulting investigations performed by the applicant, the staff sought the assistance of the contractor from the Brookhaven National Laboratory. The staff and its advisor from the laboratory visited the ESP site and met with the applicant to assist in confirming the interpretations, assumptions, and conclusions it presented regarding the characteristics of the subsurface materials and potential foundation layers. In addition, the staff and its advisor observed soil samples taken from the field explorations.

2.5.4.3.1 Properties of Subsurface Materials

In this section, the staff performed its review of SSAR Sections 2.5.4.2 and 2.5.4.3 and ER-02. In particular, the staff focused its review on the applicant's investigation results for subsurface materials, field investigations, laboratory testing, and engineering soil properties (static and dynamic) of the ESP site subsurface materials. The staff concludes that the methods used by the applicant for the exploration of subsurface materials, field investigations, laboratory testing, and determination of the engineering soil properties (static and dynamic) meet the guidelines of RS-002, and the results obtained are reasonable, except for those issues identified as a result of its review of the SSAR and responses to the RAIs, summarized in Section 2.5.4.1.1 of this report. The following discusses the staff's findings regarding the remaining unresolved issues.

From its review of the applicant's response to RAI 2.5.4-1, as summarized in Section 2.5.4.1.1 of this report, the staff finds that the revised material descriptions for the site indicate a change from the term "Catahoula bedrock" used in the UFSAR to dense sand and gravels in the current descriptions. The information from both the previous extensive site studies described in the UFSAR as well as the limited ESP investigation indicates that the foundation soil properties are consistent and that these soils are stiff enough so as to not impact evaluations of settlement and required strength. The change in nomenclature does not have any significant impact on findings in the SSAR. The staff considered the details provided with this response to be appropriate, except that the applicant did not address the basis to ascertain the layer as the Catahoula formation and correlate it with the supporting soils beneath the GGNS foundation, with only one ESP borehole reaching the depth of the soil and with very limited sampling taken inside this layer. Moreover, the staff noted that the applicant stated in its response to this RAI that the depth of new Category I foundations may have to be lower than the current GGNS foundation in order to have equivalent soil characteristics. To locate the new plant foundations lower than the existing GGNS foundations may cause significant impact on construction procedures anticipated for the site. The staff requested the applicant to evaluate the potential impact on the construction procedures and commit, in the SSAR, to implement possible techniques commonly applied in industry to prevent possible ground movement caused by deep temporary construction excavations.

In its additional response to RAI 2.5.4-1, the applicant committed to add two new paragraphs to the end of SSAR Section 2.5.4.5. In the new paragraphs, the applicant stated that if construction excavations must be lower than the existing plant foundations, excavation walls will be sloped back, reinforced (e.g., soil nails or grout mixing), or supported by temporary tied-back retaining walls similar to those used for the existing plant construction. These industry-standard deep excavation support measures will permit a safe excavation to the required foundation elevations and prevent significant ground movements. The applicant also stated that as an alternative, a combination of ground improvement or back-sloping of the excavation with the tied-back walls will be used. With regard to the ground water table, which is higher than the lower parts of the future excavations, the applicant stated that cutoff walls, collector sumps and pumps, and/or dewatering wells can be used to control the ground water. The staff's review found that the construction techniques committed to by the applicant meet the industry standard; therefore, the applicant's response to this issue is acceptable. The commitment of using excavation walls (or a combination of ground improvement with tied-back walls) and controlling the ground water during the excavations at the COL stage is **COL Action Item 2.5-1.**

In RAI 2.5.4-2, the staff requested the applicant to describe the character of the fill material and controls, if any, that were placed on the fill at the time of its deposition. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff finds that the GGNS licensee filled the original southwest-trending swales that existed in the area during the site grading associated with the prior development of the GGNS site. The fill placed at that time brought the ESP site to its current configuration. The staff also concurs with the applicant's conclusion that, since the fill does not extend to significant depths, it would not impact foundations of any powerblock facilities to be constructed in the area. The staff concludes that the procedure used by the applicant is consistent with industry practice and is acceptable. Therefore, the staff considers RAI 2.5.4-1 resolved. The applicant's commitment to conduct detailed studies of the fill material and the required treatment is **COL Action Item 2.5-2.**

From its review of the applicant's response to RAI 2.5.4-3, as summarized in Section 2.5.4.1.1 of this report, the staff found that the new borings and CPTs taken as part of the ESP investigation present site-specific information only to a maximum depth of approximately 240 feet. The applicant did not obtain detailed information about the deeper materials of the soil column for this evaluation. In addition, the staff considered the number of borings available for the ESP site from which to assess soil characteristics variability and its impact on site ground motion response, particularly at the greater depths, to be insufficient to characterize the site unambiguously, as required in Section 2.5.4.1 of RS-002. In its additional response to RAI 2.5.4-3, the applicant committed in the revised SSAR to perform additional borings, laboratory testing, and a geophysical survey to confirm the current base case material properties and their variabilities throughout the site during the COL stage. If the investigations to be performed during the COL stage indicate differences in material properties which may significantly impact on design ground motions, the applicant agreed to evaluate the need to perform additional site response analyses with the updated properties to develop updated design motions. The applicant's response is acceptable to the staff, and this issue is considered resolved. On this basis, the applicant's commitment is **COL Action Item 2.5-3.**

In RAI 2.5.4-4, the staff asked the applicant to provide the basis for selecting the generic base case velocity model as opposed to any other model that may be generated from available information for the site and its environs. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this SER, and its additional responses to the staff's comments, the staff concludes that the applicant did not justify why it applied the generic shear wave base profile based on research conducted in the Memphis area to the ESP site. As the applicant stated in its response, the embayment structure is so different in itself that the basement rock depth varies from near zero at Cairo, Illinois, to over 10,000 feet inside the Grand Gulf Basin. Memphis and its surrounding area and the ESP site belong to two completely different tectonic units—the ESP site is inside the Gulf Coast Basin and Memphis and its surrounding area is located inside the Mississippi embayment (see SSAR Section 2.5.1).

In its additional response to the staff's comments on RAI 2.5.4-4, the applicant also explained why it selected the wave velocities of 700 m/s to 2500 m/s at a depth of 1 kilometer for the profiles used in the sensitivity test. The applicant stated that it used the extreme shear wave velocities for the soil profiles. It used a low shear wave velocity of 700 m/s, which is comparable to the shear wave velocity at 656 feet depth below the San Francisco Bay Mud, and it used a high shear wave velocity of 2500 m/s, which is comparable to the shear wave

velocity for a firm rock. The staff concurs with the applicant's conclusion that the range in shear wave velocity used for the soil profile represents reasonable bounds for the sensitivity analysis. In the same response to the RAI 2.5.4-4, the applicant also explained why it provided only the sensitivity results from using the 1–2 Hz scaled spectrum, neglecting the 5–10 Hz scaled spectrum, as the control motions to the soil profiles. The applicant stated that the combined transfer function applied to the UHRS is controlled by the transfer function computed with the 1–2 Hz scaled spectrum, and the 1–2 Hz is quite similar to the UHRS and sufficiently broad band compared to the UHRS. Therefore, there is no need to perform a redundant sensitivity study using the 5–10 Hz scaled spectrum as control motions. The staff concurs with the applicant in that using the 1–2 Hz scaled spectrum is sufficient in the sensitivity test. Finally, the applicant also responded to the staff's request to address the uncertainty in kappa value, the impact of the scattering kappa value, and the sensitivity of the computed response to kappa values. As a result, the applicant revised SSAR Section 2.5.2.3. The applicant stated in this revised section that high-frequency (≥ 5 Hz) motions input to the softer portion of the profile, at a depth of about 170 feet, are sensitive to the damping in the deeper profile, which extends to hard rock conditions. This damping is constrained by the site kappa value and is taken as 0.04 seconds, a conservative estimate for this portion of the Mississippi embayment with sediment depths exceeding 10,000 feet (SERI personal communication with Professor R. Herrmann in 2002). The value of 0.04 seconds is taken as the total kappa at the surface of the loess. It includes the contribution of the low-strain damping in the hysteretic damping curves over the nonlinear portion of the profile (top 500 feet) as well as any scattering damping caused by velocity fluctuations in the profile randomization process. Sensitivity of the input motions is such that an increase in kappa to 0.05 seconds or a decrease to 0.03 seconds would result in about a 15 percent decrease or increase, respectively, in motions for frequencies exceeding about 5 Hz (EPRI, "Engineering Characterization of Earthquake Strong Ground Motion Recorded at Rock Sites," issued 1995). As a result, and because kappa can only be estimated from recordings of earthquakes, a conservative estimate of 0.04 seconds is assumed in characterizing the motions. Typical kappa values for deep soils in the western United States (WUS) range from about 0.05 to 0.07 seconds (Anderson, J.G., and S.E. Hough, "A Model for the Shape of the Fourier Amplitude Spectrum of Acceleration at High Frequencies," issued 1984, and Silva, W.M., N. Abrahamson, G. Toro, and C. Costantino, "Description and Validation of the Stochastic Ground Motion Model," issued 1997). Deep soils in the CEUS are not expected to have significantly different dynamic material properties such as shear wave velocity and material damping, particularly at depths exceeding approximately 500 feet. Based on its review of the SSAR and the response to this RAI, the staff concludes that the applicant did not provide sufficient justification for applying the kappa values derived from the observations of motions recorded in the Mississippi embayment area to the ESP site. In summary of the applicant's response to RAI 2.5.4-4, the staff concludes that the applicant needs to further justify its application of the generic shear wave velocity profile derived from the Memphis area to the ESP site and its application of the kappa value derived from ground motion observations in the Mississippi embayment to the site response calculation.

To respond to the above issues addressed in Open Item 2.5-4, the applicant provided comparison of stratigraphic section for the Gulf Coast Basin and the Mississippi embayment that shows the general stratigraphic similarity between the two areas. The extensive geological and geophysical investigations in the Gulf Coastal Plain have shown lateral uniformity of major stratigraphic groups of Cretaceous and younger age. These stratigraphic groups represent a

southward thickening sequence of marine and terrestrial deposits that were formed during a long series of marine transgressions and regressions. The applicant also emphasized that the sensitivity analysis implemented demonstrate the insensitivity of ground motions to the nature of velocity gradients likely to exist in the deep materials beneath the ESP site. On the basis of the above discussion, the staff concludes that the applicant's application of the generic shear wave velocity profile and kappa value derived from the Memphis area to the ESP site are acceptable; therefore, this open item is resolved.

In RAI 2.5.4-6, the staff requested the applicant to provide the basis for the selection of the EPRI-TR curves as opposed to other models that may be more appropriate based on site-specific information described in the geotechnical report. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff finds that the applicant used the EPRI-TR curves to represent the nonlinear properties of the three primary units of the shallow portion of the site profile (loess, Upland Alluvium, and Old Alluvium). With respect to the issue of the appropriateness of using the EPRI-TR curves to represent gravel units of the profile, the applicant's response indicates that, at the Grand Gulf site, the gravels of the profile are relatively fine gravels in a sandier matrix. These zones are also indicated to be no more than 5 feet thick and appear to be discontinuous across the site. The samples viewed by the staff during the site visit corroborate this description. On the basis discussed above, the staff considers that the use of EPRI-TR curves (soil model) to represent site soils is consistent with industry practice and, therefore, acceptable.

In RAI 2.5.4-7, the staff asked the applicant to provide the basis for not incorporating the effects of disturbance in the site response calculations and evaluate the potential impact of these effects on the computed surface UHRS. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff concludes that the applicant's procedure for selecting the EPRI-TR modulus reduction and damping curves is appropriate for the site response calculations. The process of selecting curves appropriate for depths greater than the sample depths to approximately account for overconsolidation and aging effects leads to more linear material models than are available for the EPRI-TR models. In turn, this typically leads to more conservative estimates of site response as compared to those computed from the more nonlinear models. In addition, the use of a material model randomization scheme in the probabilistic site response calculations properly accounts for uncertainty in this property of response. On this basis, the staff finds the applicant's response acceptable, and RAI 2.5.4-7 is, therefore, resolved.

In RAI 2.5.4-8, the staff asked the applicant to compare the values of shear wave velocity developed at the ESP site to the BE, UB, and LB values used in the site response calculations and explain why the mean velocity values for all the material layers are not approximately centered on the ranges listed in ER-02 Table 8.2. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that the BE shear wave velocity profile used by the applicant in the site response calculations is based on a visually averaged composite of the three P-S velocity profiles. Further, these data are not associated with specific stratigraphic units. Since the modulus degradation and hysteretic damping properties used in the calculations are also not related to stratigraphic units, the staff considers the applicant's response acceptable, and RAI 2.5.4-9 is, therefore, resolved.

In RAI 2.5.4-11, the staff asked the applicant to evaluate the impact of the velocity cutoff on the minimum depth for future siting, especially since all of the advanced reactor designs require a minimum shear velocity of 304.8 m/s (1000 fps). From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff finds that shear wave velocities measured at the ESP site fall below the target velocity of 304.8 m/s (1000 fps) at depths below those indicated in the SSAR to be probable depths for new foundations. The applicant's response also refers to these low-velocity zones at depth as localized zones. Since only three borings are available for the ESP site evaluation, one may find the shear wave velocity in these soft zones to be even lower during the detailed site investigations to be conducted during the COL stage.

In its additional response to RAI 2.5.4-11, the applicant committed to perform additional site investigations throughout the ESP site during the COL phase and to confirm that soils at the plant foundation depth have a minimum shear wave velocity of 1000 fps. The staff's review of the applicant's response thus concludes that this issue is resolved. To locate the new plant foundations on the soil with a minimum shear wave velocity of 1000 fps is a site characteristic (see Section 2.5.4.3.10 of this SER).

In RAI 2.5.4-12, the staff asked the applicant to quantify and evaluate the impact of the difference in blow counts for the SPT between previous field programs and the current program performed for the ESP site (e.g., taking a new boring adjacent to an old boring using new equipment). From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff finds that the applicant did not correlate SPT blow count data between the different hammer systems used for the ESP and UFSAR programs. However, the applicant made qualitative judgments to correlate results from the two different data sets. Based on its review experience and understanding of industry practice, the staff finds the applicant's justification acceptable. In addition, using more recent SPT results leads to generally conservative estimates of site response. On the basis of the above discussion, the staff considers RAI 2.5.4-12 resolved.

In RAI 2.5.4-13, the staff asked the applicant to explain the measures it took, if any, aside from the qualitative statements for ascertaining whether any significant disturbance occurred during the sampling, transportation, or laboratory extrusion process. Since the static testing program included CU triaxial tests, the staff expected the applicant to address whether any concern was expressed about disturbance to these samples as well. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that the applicant evaluated the potential effects of sample disturbance against common industry practice and properly incorporated these effects into its site response calculations. On this basis, the staff considers the applicant's response adequate, and RAI 2.5.4-13 is, therefore, resolved.

In RAI 2.5.4-14, the staff asked the applicant to (1) provide the basis for making the statement that the shear wave data are of excellent quality in the three boreholes, (2) indicate that the statement applies equally well to the quality of the corresponding P-wave profiles, and (3) explain the cause of the difference in P-wave velocity changes at elevations near the water table between boreholes. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that the explanations provided by the applicant with respect to the quality of compression and shear wave data are adequate since (1) the

process used to generate wave velocities used multiple measurements and (2) the process was independently reviewed. However, the SSAR must clarify the basis for the statement associated with the rise and fall in P-wave data in Boring WLA B-2A. In its additional response to RAI 2.5.4-14, the applicant agreed to add a new paragraph to SSAR Section 2.5.4.1.4, which refers to SSAR Figure 2.5-71 and justifies the statement that the rise and fall of the localized P-wave velocity may not be a result of a soft or unusually weak soil horizon. The applicant also stated that most proposed foundation excavations will be near or below this zone, such that the zone will either be removed or excavated and recompacted. Geotechnical investigations performed during the COL phase will provide additional verification regarding the soil properties of this zone with rise and fall of P-wave velocity. The staff's review finds that the justification is consistent with industry practice and is acceptable. The applicant's commitment to perform geotechnical investigations during the COL stage is **COL Action Item 2.5-4.**

In RAI 2.5.4-15, the staff asked the applicant to (1) demonstrate that the descriptors used will not mislead evaluations based on verbal descriptions, (2) provide the basis for developing soil classifications, and (3) provide the basis to justify the validation of judgments made for samples for which only one CU test was performed to estimate strength parameters (cohesion and friction angle). From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that, in general, the responses provided clarify some of the concerns raised in the RAI. With respect to the first issue, the response indicates that the applicant modified SSAR Table 2.5-7 to indicate the samples with modified descriptors. In addition, the response indicates that these modifications do not change the conclusions on potential foundation behavior. The staff finds that this response is appropriate. The response to the second issue indicates that the applicant used the procedures of ASTM D2488, associated with the use of the visual-manual procedures for field descriptions, to estimate the USC categories. This is a common industry procedure and is considered appropriate. The response to the third issue indicates that the applicant used the triaxial test results as index tests to compare strength estimates between stratigraphic layers as well as with more detailed studies performed for the UFSAR.

In RAI 2.5.4-16, the staff asked the applicant to provide the basis for the selection of confining pressures to be used in the dynamic testing and compare the resulting pressures with current estimated in situ stress levels. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that the response provided and the updated SSAR table are appropriate to explain the basis for the selection of the testing regimes for the samples provided to the laboratory. The response indicates that the comparison of the laboratory and field shear wave data is generally more appropriate for the assumed value of 1.0. This response is considered adequate and RAI 2.5.4-16 is resolved.

As discussed above, the staff found that the applicant provided a sufficient description of the subsurface material properties to support its ESP application. On this basis, the staff concludes that the subsurface materials presented in this SSAR section are acceptable.

2.5.4.3.2 Relationship of Foundations and Underlying Materials

Section 2.5.4.3 of RS-002 directs the staff to compare the applicant's plot plans and profiles of all seismic Category I facilities with the subsurface profile and material properties. Based on the comparison, the staff can determine if (1) the applicant performed sufficient exploration of

the subsurface materials and (2) the applicant's foundation design assumptions contain an adequate margin of safety. The staff concludes that the applicant's description of the relationship of foundations and underlying materials is consistent with the approach taken by industry and is, therefore, acceptable. The applicant will provide this information as part of its COL submittal. Correlating the plot plans and profiles of each seismic Category I facility with the subsurface profile and material properties to ascertain the sufficiency of selected borings to represent the spectrum of soil variations under each structure is **COL Action Item 2.5-5.**

2.5.4.3.3 Excavation and Backfill

The applicant stated that excavations can potentially reach depths of more than 100 feet below grade. It did not discuss issues such as (1) the potential extent of excavation, (2) methods of excavation and temporary support of excavation, (3) backfill sources and quality control requirements of backfill, (4) control of ground water during excavation, and (5) settlement control during excavation and construction. The applicant did not note any unusual experiences encountered during construction of the existing GGNS facilities. However, the applicant has not selected a reactor design or location within the ESP site, and it has not provided detailed excavation and backfill plans or plot plans and profiles as described in Section 2.5.4 of RS-002. These can pose special concerns for deep excavations near the bluff at the edges of the ESP site. Therefore, the staff cannot adequately evaluate the applicant's excavation and backfill plans until an applicant submits these plans as part of a COL or CP application.

The staff notes that the applicant should evaluate potential excavation procedures that may be used, as well as the impact of the adjacent bluff on temporary support conditions and on standoff distance in the ESP area. This is **COL Action Item 2.5-6.**

2.5.4.3.5 Ground Water Conditions

In its review of SSAR Section 2.5.4.2, the staff focused on the information regarding ground water at the ESP site. The applicant has not selected a reactor design or location within the ESP site and did not provide an evaluation of ground water conditions as they affect the foundation stability or detailed dewatering plans as described in Section 2.5.4 of RS-002. Therefore, the staff could not evaluate the ground water conditions as they affect the loading and stability of foundation materials, the applicant's procedure for dewatering during construction, and ground water control throughout the life of the plant. As such, the staff cannot adequately perform its review until the applicant submits these evaluations and plans as part of the COL application. This is **COL Action 2.5-7.**

2.5.4.3.6 Response of Soil to Dynamic Loading

In its review of SSAR Section 2.5.4.3, the staff primarily focused on the low-strain shear wave velocity profiles used to determine the response of the soil and rock underlying the ESP site to dynamic loading. In addition, the staff reviewed the applicant's nonlinear soil models used to incorporate the variation of soil shear modulus and damping with cyclic shear strain. Finally, the staff reviewed the applicant's site dynamic response, which it based on a soil amplification/attenuation analysis using a single base case site profile. From its review, the staff identified several issues.

In RAI 2.5.4-5, the staff asked the applicant to identify the values of the BE, UB, and LB velocities selected for each primary component of the profile and to provide bases for their selection in either SSAR Section 2.5.4 or SSAR Section 2.5.2. From its review of the applicant's response as summarized in Section 2.5.4.1.1 of this report, the staff found that the response provided by the applicant does not describe the implementation of the randomization scheme used in the response calculation. For example, it is typical to specify not only the BE velocity profile but also the corresponding ±1 sigma values of log shear wave velocities for the entire site column above hard rock, from which the randomization scheme can move forward. As discussed in Section 2.5.4.3.1 of this report, the applicant committed in the revised SSAR to perform additional borings, laboratory testing, and a geophysical survey to confirm the current base case material properties and their variabilities throughout the site during the COL stage. However, in consideration of the impact to the calculated surface ground motion from these parameters, the applicant must provide the basis for the selection of such profile properties.

In its response to the issues addressed in Open Item 2.5-5, the applicant indicated that SSAR Figure 2.5-60 shows the BE velocity profile. This profile is based on a visual average of the three P-S suspension log surveys obtained from the ESP site borings. (The applicant provided a discussion of the development of the BE profile in its response to RAI 2.5.4-8 dated December 10, 2004). Because of the small amount of data available for each unit of the BE profile, the applicant did not develop the UB and LB profiles; rather, it used a profile randomization scheme to incorporate typical values of uncertainty across the site. The applicant further stated that the profile analysis of variance uses measured shear wave velocity profiles from alluvial sites located in both the WUS and CEUS, resulting in a generic coefficient of variation (COV) of about 0.3 for shear wave velocity. Based upon experience with measured profiles in the embayment region as well as an examination of the three suspension log profiles taken at the site, as shown in SSAR Figure 2.5-80, the applicant considered the generic model unlikely to underestimate site-specific variability across the foundation footprint. Site-specific footprint (soil) variabilities typically reflect a COV closer to 0.2, but require a minimum of 20–40 profiles to reasonably constrain the model parameters.

In addition to the above discussion, the applicant provided the following information in the SSAR Section 2.5.2.4, Revision 2 of its application:

> The profile analysis of variance (Reference 194; Reference 197) used measured shear-wave velocity profiles from alluvial sites located in both the WUS and CEUS, resulting in a generic COV of about 0.3 for shear-wave velocity. Based upon experience with measured profiles in the Embayment region as well as an examination of the three suspension log profiles taken at the site (Figure 2.5-80), the generic model was considered unlikely to underestimate site-specific variability across the foundation footprint. Site-specific footprint (soil) variabilities typically reflect a COV closer to 0.2, but require a minimum of 20–40 profiles to reasonably constrain the model parameters.

Based on the applicant's response described above and its commitment to the SSAR revision, the staff finds that the selected values of 1 sigma used for the profile randomization process are appropriate, since the range of selected values plays a secondary role in generating mean site amplification factors. On this basis, Open Item 2.5-5 is considered resolved.

In RAI 2.5.4-9, the staff asked the applicant to provide its reasoning for the statement that the ESP site is not susceptible to long-term problems such as dissolution cavities and/or sinkholes. From its review of the applicant's response as summarized in Section 2.5.4.1.3 of this report, the staff concludes that karst formations are probably not of concern in the calcareous clays and limestone deposits at the site. However, the applicant further noted that additional site investigations would be conducted during the COL stage, including deep borings in the footprint of the powerblock structures. The future boring program must evaluate the potential for karst formation. This is **COL Action Item 2.5-8.**

As discussed above, the staff concludes that the applicant provided, in SSAR Section 2.5.4.3, sufficient and acceptable information for it to perform dynamic response analyses for the ESP site. On this basis, the staff concludes that the applicant's discussion of soil responses to dynamic loading is acceptable.

2.5.4.3.7 Liquefaction Potential and Seismic Site stability

As indicated in Section 2.5.4.4 of the SSAR, the soil deposits underneath the ESP range in age from Miocene (Catahoula formation) to Pleistocene (loess). These deposits all appear to be overconsolidated. The applicant does not expect to encounter any Holocene materials or relatively loose sands or silts that may be susceptible to liquefaction at the ESP site location. The applicant did not find any reported paleoliquefaction features in the ESP site vicinity. Soils below the ground water table that are planned to provide foundation support are relatively dense, overconsolidated, and relatively old and thus are not susceptible to liquefaction. The staff concludes that the applicant's description of the liquefaction potential and seismic stability of the site conditions meets with the RS-002 guidelines and is, therefore, acceptable.

2.5.4.3.8 Static Site Stability

In its review of SSAR Section 2.5.4.5, the staff focused on the applicant's evaluation of bearing capacity, potential of settlement, and lateral earth pressure. In SSAR Section 2.5.4.5.2, the applicant stated that several inches of elastic rebound may be associated with relatively deep site excavations and that it expected this rebound to be reversible, presuming that the new structures are fully compensated designs. Assuming that the current GGNS powerblock structures are far enough from the ESP site, the staff requested, in RAI 2.5.4-10, the applicant to demonstrate that no other facilities (e.g., piping, conduit) exist in the ESP area that may be influenced by such surface movements.

The staff reviewed the applicant's response as summarized in Section 2.5.4.1.5 of this report and found that the applicant stated, in SSAR Section 2.5.4.5, that the anticipated settlements associated with the ESP construction will be relatively small (several inches). This is based upon a review of the known soil conditions at the site as well as settlements recorded during construction of the existing GGNS powerblock structures. In addition, no safety-related facilities exist in the zone of influence of the ESP construction. Since the ESP site is immediately adjacent to the existing GGNS site, the staff considers the response provided by the applicant adequate, and RAI 2.5.4-10 is resolved.

On the basis of the above discussion, the staff concludes that the applicant's evaluation of bearing capacity, potential of settlement, and lateral earth pressure for the ESP site meets the RS-002 guidelines and is, therefore, acceptable.

2.5.4.3.9 Design Criteria

In SSAR Section 2.5.4.6, the applicant stated that specific design criteria will be developed during the COL stage when the specific characteristics of the operating system are known. Design criteria associated with structural design, such as potential wall rotations, facility sliding, and overturning, must be developed for specific facilities. Therefore, this is **COL Action Item 2.5-9.**

2.5.4.3.10 Site Characteristics Related to Geotechnical Engineering

Based on its review of SSAR Section 2.5.4, the staff has determined that the site characteristic given in Table 2.5.4-1 should be included in any ESP that might be issued for the proposed site.

Table 2.5.4-1 Staff's Proposed Site Characteristics Related to Geotechnical Engineering

SITE CHARACTERISTIC	VALUE
Minimum Shear Wave Velocity of Soil at Plant Foundation Level	1000 fps

2.5.4.4 Conclusions

Based on its review of SSAR Section 2.5.4 and the applicant's responses to the associated RAIs described above, the staff concludes that the applicant adequately determined the engineering properties of the soil encountered during its field and laboratory investigations, and that the applicant has provided sufficient technical information in the geotechnical area to demonstrate the suitability of the ESP site for building a new nuclear power plant, except for those areas for which more information is needed to allow the staff to make its decision regarding the site. In addition, the applicant used the latest field and laboratory methods, in accordance with RGs 1.132, 1.138, and 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites," issued November 2003, to determine these properties. However, the staff also concludes that the applicant did not perform sufficient field investigations and laboratory testing to adequately define the overall subsurface profile, as well as the potential variability of the properties of the soil underlying the ESP site. The staff notes that the applicant committed to perform additional field investigations once it has selected the locations and facilities for safety-related structures at the COL stage.

2.5.5 Stability of Slopes

Section 2.5.5 of the SSAR discusses issues associated with slope stability at the ESP site.

2.5.5.1 Technical Information in the ESP Application

In SSAR Section 2.5.5, the applicant stated that the ESP site is relatively flat and will not be subject to large-scale landslides or slope failures. The location of the proposed new facility

encompasses two flat graded surfaces that are separated by an approximately 22-foot-high, 3:1 (20-degree slope) cut-slope in loess soils. The cut-slope has shown no evidence of instability since its construction in the early 1970s. At the west side of the ESP, the area is bounded by a 65-foot-high erosional escarpment (bluff) that descends to the Mississippi River floodplain. Portions of the bluff are subject to surficial slumps and creeping soils that are confined within the loess soils in the face of the bluff slope. Based on its observation of no evidence of active or incipient slope movements above or around the possible slump scarp on the bluff, the applicant stated that future instability in the bluff slope should not affect the future facilities because the surficial slumping and erosion in the bluff slope are restricted to the loess soils. The applicant also stated that it is likely that the future plant footprint will be located at least 100 feet from the top of this possible slump feature and the plant foundations would extend through the loess soils into the underlying Upland Complex Alluvium or Old Alluvium, which is well below the possible slide planes or toe of the bluff slope.

As stated in the SSAR, no plans exist to perform specific analyses to evaluate the stability of deep foundation excavations for the future facilities at the proposed ESP site until the COL phase. In addition, the applicant stated that it did not find any indications of unstable materials underlying the site that should cause unusual stability conditions for excavations and excavation support. In SSAR Section 2.5.5 and Sections 3.3 and 12.5.2 of ER-02, the applicant noted that a 60–70-foot escarpment that may be subjected to surficial slumps and potential creep of the loess soils bounds the west side of the proposed ESP site. The applicant further stated that, since future safety-related facilities would be founded on Upland Alluvium or Old Alluvium encountered below the loess, future movements of the slope should not have any significant impact on these foundations. However, if such facilities are founded close to the scarp, such slump or creep effects could have an impact on lateral loads applied to such deeply founded facilities. In RAI 2.5.5-1, the staff asked the applicant if it had performed any evaluation to indicate the expected behavior of the loess escarpment or the extent to which such movements could occur. In addition, the staff also asked the applicant if the ESP site should have some exclusionary zones along the west-side boundary that would not be susceptible to such potential future slump. In its response, the applicant stated that SSAR Figure 2.5-69 shows the spatial relationships among the PPBA and the loess bluff and the postulated shallow slump in the bluff near the west boundary of the ESP site. The applicant evaluated the potential hazard to the site from slope failure and creep in the loess bluff by examining geologic cross-sections, evaluating embedment depths and positioning of ESP structure foundations, and qualitatively assessing the bluff slope stability in relation to the relative strength of subsurface materials. The applicant further stated that it developed a simplified cross-section to show the geometric relationships between the loess bluff and the site. In this schematic cross-section, the PPBA is set back over 100 feet from the closest approach of the bluff and top of the postulated slump. The maximum foundation influence zone envelope extends to just within the headscarp of the slump area. However, the likely depth range for the prospective reactor foundations required to satisfy minimum embedment and shear wave velocity criteria places the foundation level about 10–80 feet below the elevation of the toe of the bluff slope. The applicant also stated that it is unlikely to develop a failure plane that extends from the toe of the bluff or headscarp of the postulated slump back to the PPBA. A failure plane would have a very gentle inclination of about 8 degrees and would need to intercept the perimeter of the PPBA at least 5–10 feet below plant grade. The applicant believed that such a low-angle failure plane is very unlikely based on an estimated in situ effective friction angle of 33–34 degrees measured by triaxial UC

testing of loess samples. Therefore, the applicant concluded that no kinematically feasible bluff failures could extend back to, or negatively impact, the stability and lateral confinement of prospective reactor foundations within the PPBA. Based on the qualitative stability assessment, the hazard to the ESP site from possible future movements in the loess bluff is very low to none and does not require additional analysis at this ESP stage. The applicant stated that an exclusionary zone between the top of the bluff and the ESP PPBA, therefore, is not necessary.

2.5.5.2 Regulatory Evaluation

SSAR Section 2.5.5 states that the applicant did not analyze the stability of existing slopes since the foundations of safety-related facilities will be located below the soils that may be susceptible to stability problems. As such, the applicant did not list any regulatory guidance or cite any regulations as applicable to this section of the SSAR.

In its review of the application, the staff considered the regulatory requirements in 10 CFR 100.23, which states that the applicant for the ESP must describe the geologic and seismic conditions of the proposed site necessary to determine the site stability. Section 2.5.5 of RS-002 provides specific guidance concerning the evaluation of information characterizing the stability of slopes under SSE conditions.

2.5.5.3 Technical Evaluation

In RAI 2.5.5-1, the staff requested the applicant to perform an evaluation to demonstrate the expected behavior of the loess escarpment or the extent to which such movements will not occur. In its response, the applicant noted that it modified the ESP site plan to restrict the location of the PPBA to a distance of over 100 feet from the bluff area on the west side of the site. The applicant also stated that, based on a qualitative assessment of stability, the hazard to the ESP site from potential future movements of the loess soils is very low to none. However, this qualitative assessment was based on potential failure plane relationships and did not consider the potential impact of differences in elevations on soil-structure interaction (SSI) evaluations of safety-related facilities. In its additional response to RAI 2.5.5-1, the applicant committed to add a new paragraph at the end of SSAR Section 2.5.4.3. In this new SSAR section, the applicant referred to Figures 2.5.5-1 through 2.5.5-2, reproduced below, and provided an assessment of possible behavior of the site topography. The applicant summarized that the possible future slumping or erosion could result in some changes in the local topography, but this should not result in a measurable reduction of soil lateral capacity for plant structures located in the prospective powerblock area, or significantly influence the lateral response of the soils under dynamic loading from the buried structure at the edge of the site. With regard to the potential impact of differences in elevations on SSI evaluations of safety-related facilities, the applicant committed, in the new SSAR section, that the COL applicant will incorporate the effects resulting from the local topography or possible changes in topography in the SSI analyses to be performed for structures located in the southwest quadrant of the ESP site. The staff's review found the commitment made by the applicant in its RAI response acceptable. To incorporate the effects resulting from the local topography or possible changes in topography in the future SSI analyses is **COL Action Item 2.5-10.**

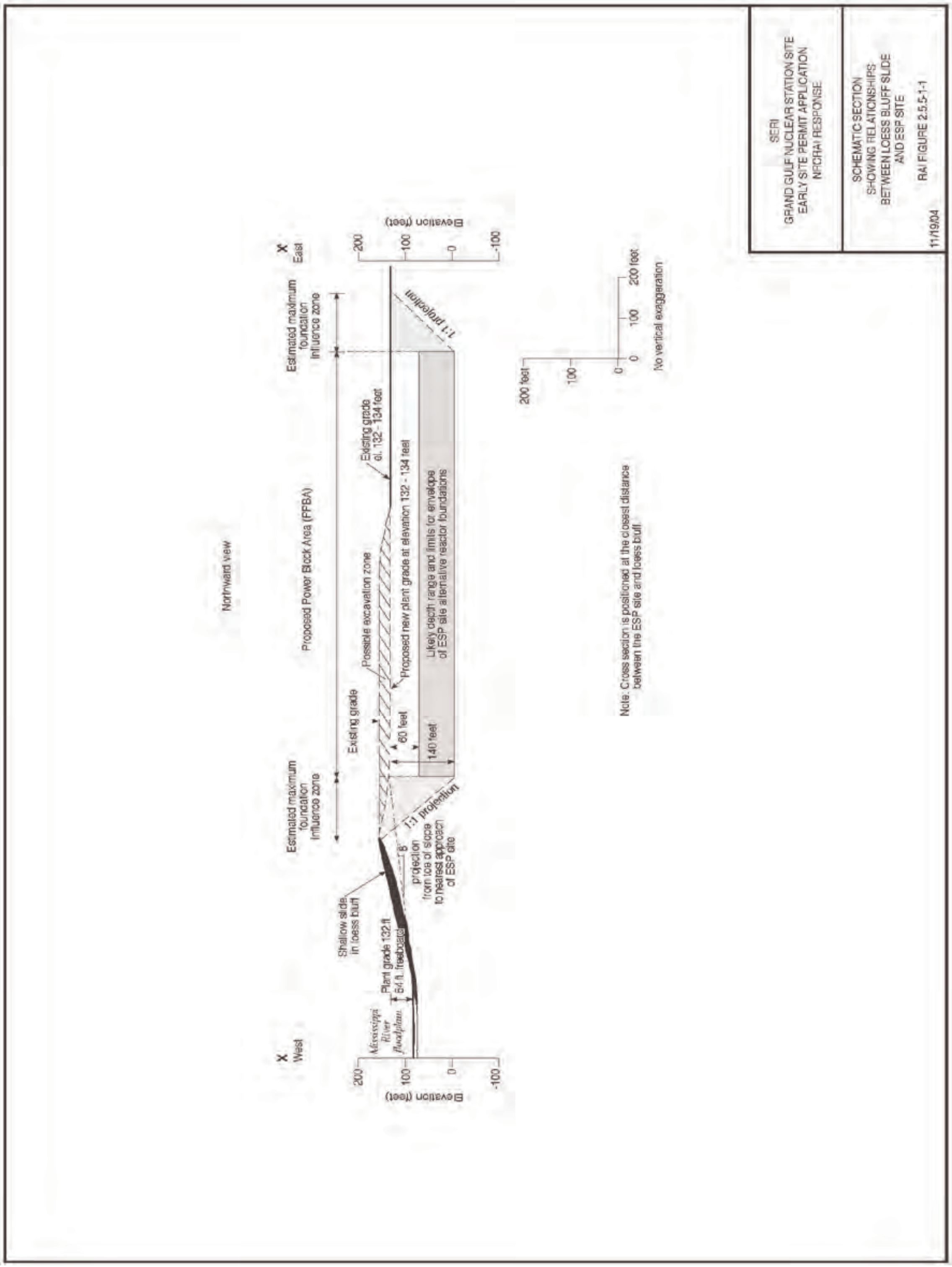


Figure 2.5.5-1 (RAI Figure 2.5.5.1-1)
Schematic Section Showing Relationships Between Loess Bluff Slide and ESP Site

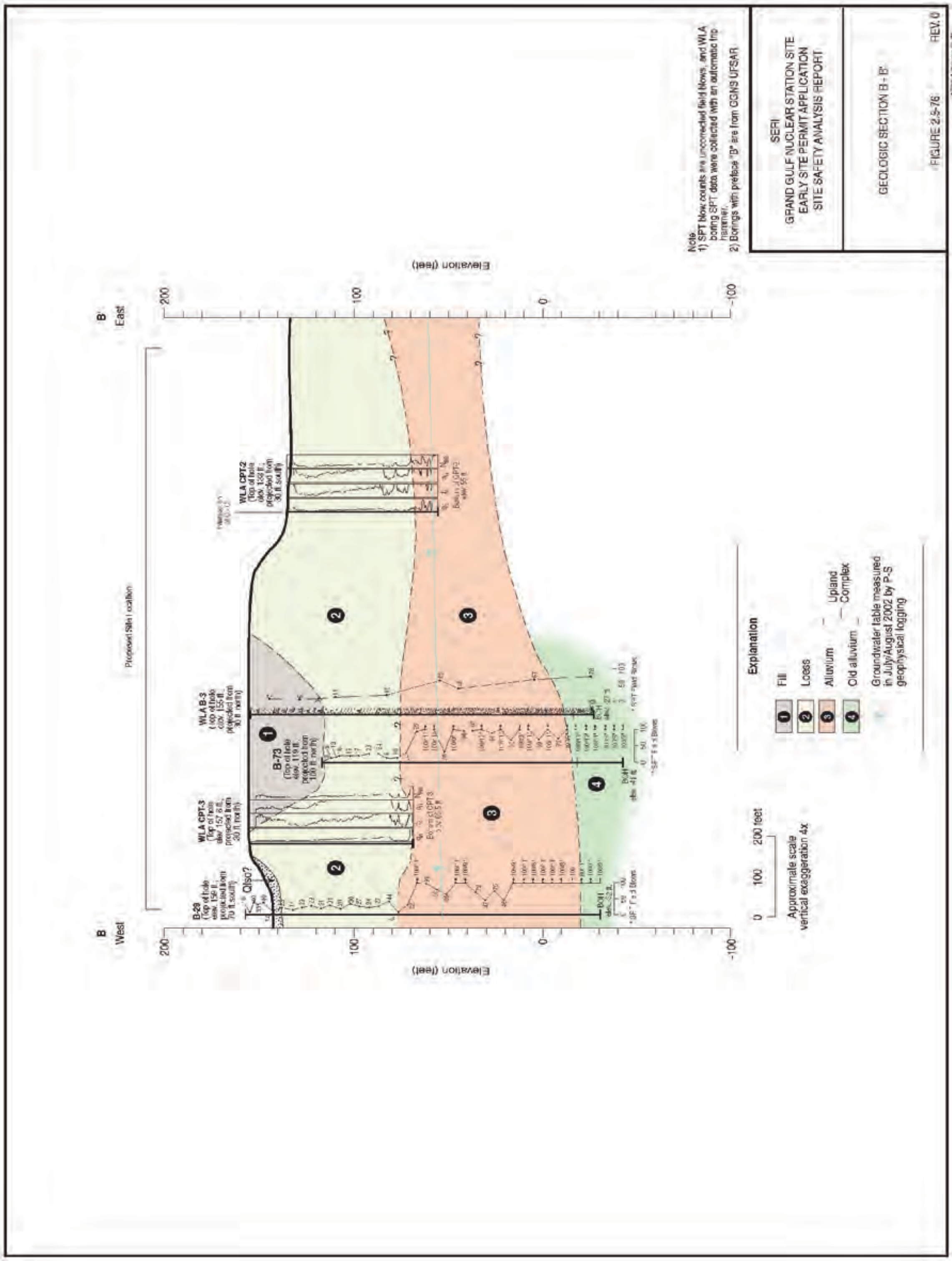


Figure 2.5.5-2 (SSAR Figure 2.5-76)
Geologic Section B - B

As discussed above, the staff found that the applicant provided sufficient description of the stability of slopes in the area to support its application for the ESP. On this basis, the staff concludes that the slope stability assessment presented in this SSAR section is acceptable.

2.5.6 Embankments and Dams

SSAR Section 2.5.6 considers embankments and dams associated with the ESP site.

2.5.6.1 Technical Information in the Application

SSAR Section 2.5.6 indicates that, within the ESP site area, no earth or rock fill embankments are used for flood protection or impounding the cooling water. In addition, no impoundment structures within the ESP site area exist that could pose a hazard to the proposed future facility. Therefore, no significant hazards may be posed by inundation from such facilities. The SSAR does not indicate any influence of flooding from the Mississippi River and its potential to further erode the loess bluff.

2.5.6.2 Regulatory Evaluation

Since the applicant stated that no impoundment structures lie within the ESP area, the applicant did not list any regulatory guidance or cite any regulations as applicable to SSAR Section 2.5.6. Section 2.5.6 of RG 1.70 describes the necessary information and analysis related to the investigation, engineering design, proposed construction, and performance of all embankments used for plant flood protection or for impounding cooling water. RS-002 Sections 2.4.4, and 2.5.5 provide similar information and guidance.

2.5.6.3 Technical Evaluation

The staff's review found that, although no impoundment structures lie within the ESP area, the applicant did not evaluate the effect of potential flooding of the Mississippi River and possible future erosion of the bluff. The COL applicant should evaluate these effects and their impacts on SSI effects. This is **COL Action Item 2.5-11.**

2.5.6.4 Conclusions

The staff found that the applicant provided sufficient descriptions of the embankments and dams in the site vicinity to support its ESP application, except that the applicant must provide additional information to address the COL action item. On this basis, the staff concludes that the assessment of embankments and dams presented in this SSAR section is acceptable for the ESP application.